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Structural Performance of Prefabricated Straw Bale Panels

Christopher Gross

A thesis submitted for the degree of Master of Philosophy

University of Bath
Department of Architecture and Civil Engineering

July 2009

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Abstract

The public and construction industry professionals are becoming increasingly aware of the environmental damage buildings can cause. As a result the use of more sustainable and environmentally friendly building materials and techniques is increasing. Straw bale construction is one such technique. ModCell panels are an innovative way of using straw bales in a prefabricated wall system. As ModCell panels are prefabricated the normal risks associated with straw bale construction (fire, straw getting wet during construction, unfamiliar material) are removed from the building site. The panels consist of a timber frame, braced with steel rods, in-filled with straw bales and the surfaces are then rendered with a lime render. To date ModCell panels have been used for cladding on framed buildings. However, they are now being developed for low rise domestic load-bearing applications and in order to achieve this sufficient racking shear resistance is required. Previous testing has shown that the corner joints in the panel’s timber frame are a weak point.

This first aim of this thesis is to improve the racking shear resistance of ModCell panels through a series of experimental laboratory based tests so that they can be used for load-bearing applications. This thesis also aims to develop a computer model of load-bearing ModCell panels in order to further understand their structural behaviour and assist with future design and development. Different designs of frame corner joints were tested in order to find the most suitable type of joint. Screw connected joints proved to be the strongest and were twice as stiff as the other joint designs tested. Full scale panel racking shear tests were undertaken to assess the overall performance of the panels when using the screw connected joints and establish if they are suitable for load-bearing construction. Both solid panels and panels with window openings were tested. Two types of bracing were used in the test panels, either corner bracing or cross bracing. The corner braced panels were over twice as stiff as panels tested during a previous research project. The corner braced panels were also stiffer than the cross braced panels at the serviceability deflection limit of the panels height/500, and was sufficiently stiff to resist the required in plane loading under serviceability conditions.

The computer model was developed alongside the structural testing. The model was developed using the Robot Millennium software package and modelled all elements of the panels apart from the straw. It was verified against the laboratory test results and found to be accurate within the serviceability deflection limit of height/500. Once material failure occurred in the actual panels the model's accuracy was reduced as it does not include material failure criteria. The model was then used to undertake a parametric analysis. From this analysis it was found that all the elements modelled have an effect on the stiffness of ModCell panels. Changing the render thickness was found to make the greatest difference on the overall stiffness of the panels. Gusset plate and glued and screwed joint designs were also modelled to find their effect on panel stiffness. From all of the results of this analysis an improved panel was designed and when modelled was 20% stiffer than the original panel.

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1 Introduction

1.1 Background

In England and Wales the construction industry produces 109 million tonnes of waste annually. It is estimated that 13% of this waste is new material that is never used on site (Defra 2007). Approximately 50% of the UK's CO_2 emissions are from construction and the running of buildings (Dti 2004) and 5% of global CO_2 emissions are from cement production alone (Morton 2008).

Over recent years the construction industry and the general public have become increasingly aware of this environmental pollution. As a result the Dti (2004) published guidance on sustainable construction with its key themes being to design for minimum waste and minimum energy use. Many sustainable building materials satisfy both of these themes and this combined with clients and designers desire to use them is increasing their popularity. Straw bale construction is one such material, and its use within the United Kingdom is slowly increasing.

1.2 Brief history of straw bale construction

Straw has been used for thousands of years as a building material. Traditionally straw was mixed with clays in earth construction techniques such as wattle and daub, cob and adobe in order to reinforce the earth (King 2006). During the late 1800s baling machines were invented in the USA which allowed farmers to store their straw more easily. At the same time communities were starting to populate and farm areas of Nebraska. Conventional building materials were in very short supply within these new communities so farmers started to use the bales produced from their crops to build houses. These buildings had load-bearing straw bale walls which were then rendered both inside and out. This type of straw bale construction has since become known as Nebraskan style and some of these buildings survive to this day (Jones 2002). Figure 1.1 shows one such building that was constructed in 1905 and is still occupied to this day (Wanek 2003). This technique of building slowly died out once other building materials become more widely available. There was a revival of this building technique in the USA in the 1980s when some people started to become concerned with the impact on the environment construction materials and housing were having (King 2006).

Figure 1.1 Nebraska style straw bale house dating from 1905 (Wanek 2003)

There are two main types of straw bale construction, load-bearing (Nebraska style) and non-load-bearing infill (Chiras 2000). In load-bearing construction typically the straw bales are placed on a timber sole plate that is fixed to a foundation slab. The bales are laid in running bond and pinned together with timber or steel stakes. The wall is capped with a timber wall plate onto which the roof structure is fixed and the bales are rendered. In this type of construction all vertical and shear loads are carried by the bales themselves. Due to the relatively low strength of this type of wall the buildings are generally only single storey.

Non-load-bearing straw bale construction consists of a structural timber frame. Straw bales are simply placed between the frame members and rendered to create solid insulating walls. In this type of construction all of the loading is carried by the frame. This allows for larger buildings with several floors. Figure 1.2 shows a house built using this technique in 1995 (Steen and Steen 2000). Within the UK prefabricated straw bale construction has been developed. This is discussed in greater detail later in this report.

Figure 1.2 Straw bale house in USA dating from 1995 (Steen and Steen 2000)

1.3 Straw bale construction in the UK

The first straw building in the UK was built in 1994. Most straw bale buildings built in the UK have been one off projects built by individuals who have an interest in building with straw. This trend is reflected in the fact that between 1994 and 2002 only about 70 straw bale buildings were constructed in the UK (Jones 2002).

There have been several notable straw bale buildings constructed in the UK. One of these is the Centre for Alternative Technology (CAT) theatre (Figure 1.3). The building was constructed in 1999 using a larch frame with straw bale infill walls. Both faces of the straw were plastered using a 50 mm thick lime plaster (Anonymous 2007).

Figure 1.3 Centre for Alternative Technology Theatre

The Straw Pavilion at the Genesis Project in Somerset was completed in 2006 (Figure 1.4). The building is rectangular in plan, has a flat roof and contains three seminar rooms. The structure is a hybrid of load-bearing straw bale walls with an internal timber frame to allow an exact final wall height to be achieved (SCAT 2007). The straw bale walls were constructed following established techniques and information from the results of structural testing by Walker (2004). The bales are pinned with hazel spikes and are pre compressed between the sole plate and timber ring beam. The Straw Pavilion is part of the larger Genesis Project Centre building which was short listed for RIBA Public Building awards 2007.

Figure 1.4 Genesis Project Straw Pavilion (SCAT 2007)

The York EcoDepot (Figure 1.5) is the largest straw bale building in Europe enclosing 1129 m^2 of floor space (ModCell 2009). The building was constructed for York City Council in 2006 using ModCell panels. The structure consists of a laminated timber frame with a light weight concrete floor deck. The ModCell panels were fixed to the frame as cladding. The building has received several awards including one from the Institution of Civil Engineers (Yorkshire and Humberside) (ModCell 2009).

Figure 1.5 York EcoDepot

Amazonails have been involved in the construction of over 40 straw bale buildings in the UK (Amazonails 2001). One of the recent projects Amazonails were involved in was the Footprint project in Cumbria. The building uses a variety of natural and recycled materials including straw bales, oak shingles, used car tyres for foundations and sheep's wool for insulation. Load-bearing straw bale walls form the main structural element of the building.

Walker (2007) investigated the barriers to increased straw bale construction in Scotland and noted that durability, fire resistance and structural performance were concerns of construction professionals. Additionally Walker (2007) suggests that a lack of education amongst designers and constructors is also a major barrier. It can be safely assumed that the same barriers will be present over the entire of the UK. As a result there is reluctance within the UK building industry to use different and innovative materials as they are an unknown quantity.

1.4 ModCell straw bale panels

ModCell panels are prefabricated straw bale panels used for the construction of low rise domestic and public buildings. ModCell panels were developed by White Design and Integral Structural Design in order to try and move straw bale building into the mainstream construction sector and make straw bale construction more easily accessible to designers, clients and constructors. The key idea behind ModCell panels is that they are prefabricated off site in a local 'flying factory' and then transported to site and constructed into a building. This removes the presence of straw bales onsite and the associated problems (storage, weather protection, fire risk).

A ModCell panel consists of a laminated timber frame, straw bales, stainless steel reinforcement and lime render. Figure 1.6 shows a typical ModCell panel.

Figure 1.6 Build up of ModCell panel

ModCell panels were developed almost 10 years ago as feature cladding panels for the University of West of England School of Architecture (ModCell 2009). There are two ModCell panels at each end of the building (see Figure 1.7). Since then there has been constant development of the panels and they have been used as cladding on several buildings including York EcoDepot (Figure 1.5) and the Knowle West Media Centre (Figure 1.8).

Figure 1.7 University of West of England

Figure 1.8 Knowle West Media Centre, Bristol

In May 2008 ModCell panels were used structurally for the first time as the load-bearing ground floor for “The House That Kevin Built” on Grand Designs Live (Figure 1.9). It is this application of ModCell panel that this investigation focuses on.

Figure 1.9 “The House That Kevin Built”

1.5 Aims and objectives

There were two key aims of this investigation. Firstly to establish the racking shear resistance of ModCell panels in order to ascertain if they can be used in two or three storey load-bearing construction. This will be ascertained through a series of experimental laboratory tests. Secondly to develop a computer model of load-bearing ModCell panels in order to assist with future development and design of the panels.

In order to assess and develop the racking shear resistance of the panels both joint testing and full scale racking shear tests were carried out. In plane racking shear loads need to be resisted by the panels in order to meet the wind load criteria. The total racking shear load on a 6.8 metre by 6.8 metre two storey ModCell house caused by the effects of wind is in the order of 35 kN (see section 3.5).

The racking shear load needs to be resisted with a maximum horizontal deflection at the top corner of the panels of less than h/500 per storey. Vertical loading is not being considered as in load-bearing ModCell construction it is transferred through the timber frame, the strength of which can be quantified using published design standards. Out of plane loads caused by the effects of wind are not being investigated as these were previously investigated by Lawrence et al. (2009a) and the panels were found to be adequate.

A computer model of load-bearing ModCell panels was developed alongside the laboratory testing and the results from the racking shear tests used validate it. This model was developed to further the understanding of the structural behaviour of ModCell panels and to aid in the future development and design of the panels.

1.6 Layout of thesis

This thesis reviews the significant testing that has been carried out to date on the load capacity of straw bales, the racking shear resistance of straw bale walls and the testing specific to ModCell panels. This thesis then outlines the laboratory testing carried out to assess the structural performance of the panels and presents discussion and analysis of the results. This thesis also discusses the development of the computer model, its validation, use in further understanding the panels and the modelling of a future load-bearing ModCell buildings.

2 Literature Review

2.1 Introduction

This section of the thesis reviews significant structural testing carried out on straw bale walls. Straw bale buildings are generally constructed following the same techniques as used by the first users of straw bales in the 1800s. This is an acceptable way of building, but it does not necessarily use the materials as efficiently as possible or to their full potential. In order to exploit them to their full potential structural testing needs to be undertaken on the materials used as well as full size wall assemblies. In the case of ModCell panels a standardised system is being developed. Often straw bale designs undergo structural testing prior to construction in order to satisfy Building Control. A standardised system will remove the need for this.

In the following chapter the straw and render used for the construction of ModCell panels will be reviewed. The load carrying capacity of individual straw bales is reviewed as the individual strength of straw bales is likely to affect the overall strength of straw bale walls. Finally significant laboratory tests on the racking shear resistance of straw bale walls are reviewed as this is the structural aspect of ModCell panels this thesis is focusing on. As the modern use of straw bale construction is still in its infancy there has only been a limited amount of research into the structural performance of straw bale walls. Much of this work has been undertaken as undergraduate and postgraduate research and has not been published in peer review publications. As a result it is difficult to obtain copies of some research or assess their reliability. For this reason the literature review in this chapter is concise, but it does cover the most significant structural testing undertaken.

2.2 Material review

2.2.1 Straw

Straw is a widespread agricultural by product. It is the stalk that supports the head of grain crops during growth. When harvested the head and stalk are separated, with the head often being used in food production and the stalk or straw is a major by product of this process (Cripps et al. 2004). Table 2.1 shows the amount of cereal crops grown within the UK in 2008. With so much production there is great potential for the straw to be used within the construction industry.

Table 2.1 Production of cereal crops in UK in 2008 (Defra 2009)

Cereal	**Area (thousand hectares)**	**Production (thousand tonnes)**
Total	3274	24282
Wheat	2080	17227
Barley	1032	6144
Oats	135	784
Minor cereals	27	128

Following harvesting straw is baled to allow for convenient storage. Originally, the process of baling left straw in a form that could easily be stacked and therefore used for construction. As the size and amount of machinery used in farming has increased so has the size of the bales. Therefore re-baling of large bales into smaller bales is often required to get a size suitable for the construction of walls. In the USA rice straw is most commonly used for construction due to its high silica content which resists decay and its rough surface texture which helps the straws bind to each other when baled (King 2006). There is little to choose between straw from other cereal crops and as rice straw is not grown in the UK the most easily available straw bales are used. This is most commonly wheat as it is produced in the largest quantities in the UK.

The bale orientation is important when being used in the construction of walls. Figure 2.1 shows a bale laid flat and a bale laid on edge. A bale laid flat is loaded perpendicular to its largest face and the individual pieces of straw generally run horizontally. A bale laid on edge is rotated 90^o along its long axis from the bale laid flat. It is loaded parallel to its largest face and the individual pieces of straw generally run vertically. Vardy and MacDougall (2004) tested plastered bales laid flat and on edge in compression. The two long sides of the bales parallel to the direction of load were plastered in two layers. The first layer used a cement lime plaster and the second layer used a lime plaster. The authors found that the plaster skins failed differently on the two orientations of the bales. On the bale laid flat the plaster failed by crushing where as on the bale laid on edge the plaster failed as a result of buckling. This is attributed to the bond between the plaster and bale not being as strong when the bale is laid on edge. Overall Vardy and MacDougall (2004) found that the compressive strength of the bales laid on edge was 36% lower than those laid flat and they conclude that this is due to the different failure modes. From the work it can be concluded that bales should be laid flat in order to get the best plaster adhesion and hence strength which is important in load-bearing straw bale walls.

The bale direction within the wall is also important. Jones (2002) and King (2006) note that the direction of the bales should be alternated within a wall as during the baling process one edge of the bale gets cut by the mechanical baling machine which allows for improved adhesion of the plaster as it can penetrate the cut ends of the straw. This is one of the reasons why during the construction of ModCell panels both the long sides of the bales are trimmed to give an even cut face which allows improved plaster adhesion.

Figure 2.1 Bale orientation (adapted from Vardy & MacDougall 2004)

King (2006) notes that the moisture content of the bales at the time of construction should be taken to check it is within acceptable limits. Lawrence et al. (2009b) state that at a moisture content above 25 %H_2O is a cause for concern as decay can begin.

2.2.2 Render

The render on straw bale buildings plays a crucial role in preserving the straw, protecting it from the weather, fire, pests and accidental damage, while remaining vapour permeable (King 2006). The render must be vapour permeable to allow the straw to release moisture and return to equilibrium when moisture does penetrate the render.

Both Jones (2002) and King (2006) suggest that reinforcement mesh can be used within the rendered surface in order to increase its strength. Much research has been carried out on walls with normal render and with reinforced render. Ash et al. (2003) found that load-bearing straw bale walls with unreinforced render failed at lower loads than those that were reinforced. But it was noted that this was related to the way the reinforcement helped to fix the render to the head plate at the top of the wall and not a failure in the render surface as a whole. There is no research that suggests that reinforcing plaster reduces the strength of straw bales walls.

The thickness of render required is a balance between that required for strength, longevity of the straw and the cost. Generally the thicker the render, the higher the load that can be carried and the greater the protection offered from the weather. But render is an expensive part of the walls and therefore the thicker it is the higher the cost. ModCell panels use 30 mm thick render as this has been found, through testing carried out by The University of Bath, to provide adequate protection from the weather.

There are three main groups of render that can be used on straw bale walls. These are earth, lime and cement based renders. Lime and cement are often combined to give a render with the best properties from each. The lime provides flexibility and vapour permeability while the cement provides fast developing strength. Faine and Zhang (2002) compared the load-bearing capacity of earth plastered and cement plastered straw bale walls. They constructed two straw bale walls; one two storey wall with internal steel bar pinning, mesh reinforcement and cement render and; one single storey wall with 3% pre-compression and earth render. The single storey wall did not contain pinning or mesh. With a 40kN working load applied the cement rendered wall deflected an average of 5mm and the earth rendered wall deflected an average of 8mm showing that the cement rendered wall is stiffer. The authors note that the earth rendered wall displayed more visible signs of weakness with severe cracking and spalling of the render. The authors also noted that the earth render was much easier to apply and that chicken wire was not required to cover the bales prior to rendering.

Render can be applied to straw bale walls either traditionally by hand or by using a render spraying machine. Spraying the render is much faster and appears to give better penetration of the render into the straw. However no testing has been under taken to prove increased adhesion.

2.3 Load carrying capacity of Straw bale walls

The load carrying capacity of straw bale walls is determined by several factors. These are:

- Bale density
- Render strength and thickness
- Render reinforcement
- Other reinforcing within the wall
- Head and sole plate details

With all of the above working compositely, straw bale walls exhibit complex behaviour when loaded. The structural model of a straw bale wall is often simplified down to a sandwich panel arrangement with two stressed skins with a soft insulating core (Carrick and Glassford 1998).

Zhang (2002) tested plastered and un-plastered individual bales laid flat and on edge and applied loads to only the straw and to the straw and render together in order to investigate the path of the load. During testing of the straw bales Zhang (2002) found that initially upon loading the bales compressed by 3 to 4% of their original height. It is common practice to pre-compress straw bales walls prior to plastering to avoid this settlement the first time the wall is loaded (King 2006). The straw bales in ModCell panels are pre-compressed by a load of roughly 2.5 kN/m which is equivalent to an average settlement of about 1.5%. This is less than the initial settlement noted by Zhang, but the bales in ModCell panels do not bear any direct vertical load and therefore settlement is not as critical.

Walker (2004) tested straw bale walls in vertical compression. The wall panels were 2.25 m high by 0.99 m wide with the bale direction alternating as recommended by Jones (2002) and hazel spikes to pin the bales together. Five different specimens were tested, one standard/control specimen, one without hazel spikes, one using half bales in every other course, one that was pre-compressed and one that was lime rendered.

The lime rendered wall carried a maximum load of 41.1 kN (41.5 kN/m) with a settlement of 55 mm. This was over 50 times stiffer than the un-rendered wall specimen. The pre-compressed wall panel had roughly half the settlement of the standard wall panel at maximum load. This research has shown that render is key to improving the stiffness and strength of straw bale walls under vertical loading and again that pre-compression reduces deflections under load.

Vardy (2009) developed a theoretical model to predict the compressive strength of straw bale walls and compared the results with results from laboratory based experiments. The author adapted a stress strain model for concrete for use with lime-cement render. The model was verified against cylinders of the lime-cement render. The model uses the render strength and thickness to predict the strength of the straw bale walls, with any compressive resistance from the straw being ignored. In total 18 wall specimens were constructed and tested in order to verify the results of the model. Three straw bale walls 0.99m high and three straw bale walls 2.32m high were pre-compressed by an unspecified amount and rendered with lime-cement render in three layers. The render was not allowed to cure between layers. The specimens were tested seven days after rendering. They were loaded with a concentric vertical compressive load. Vardy (2009) found that the model over predicted the strength of the 0.99m high walls by an average of 14% and under predicted the strength of the 2.31m high walls by an average of 10%. However, overall it was found that the model was 99% accurate. Vardy (2009) also tested walls with eccentric loads and achieved similar comparisons with the model. This is the first piece of work of its kind where a model has been developed to predict the behaviour of straw bale walls. It has shown that it is possible to accurately predict their behaviour based on a simple model of the render.

2.4 Racking shear resistance of straw bale walls

Carrick and Glassford (1998) tested straw bale walls subject to vertical and in plane racking loading. The test walls were constructed using rice straw bales and were 3.6 metres wide by 2.7 metres tall with a pre-compression of 125 mm. The wall panels were rendered with a 30 mm thick sand and cement render reinforced with chicken wire. A 10 kN (2.78 kN/m) racking load was applied to two different wall specimens and horizontal displacements of 2.3 mm and 2.4 mm were recorded. Additionally the walls were loaded in vertical compression to failure. Ultimate loads of 21.4 kN/m with 4.5 mm deflection and 20.6 kN/m with an unknown deflection were recorded. The investigation by Carrick and Glassford (1998) is interesting as they applied both vertical and racking loads at the same time, which is likely to be the situation in a load-bearing straw bale building. Unfortunately they did not test the walls with the vertical and racking loads applied independently from each other, so a comparison between combined and individual performances is not possible. They did however load one of the wall panels before rendering and concluded that the render skins provide most of the wall's strength, but the straw is also important as it prevents the render membrane buckling.

Nichols and Rapp (2000) tested a single 2.3 m high load-bearing straw bale wall with horizontal in plane loading three times over a period of two weeks. The wall was two bales wide by six bales high, covered in reinforcing mesh and plastered with two layers of plaster. The authors do not give details about the type of plaster. The first test was carried out 14 days after the final coat of plaster had been applied. Cracks formed in the plaster at a load of 98 kN and a displacement of 12.9mm (height/180). The second test was carried out two days later and the wall was not repaired between tests. The third test was carried out a further seven days later and again the wall was not repaired. During the third test at a displacement of 12.9 mm again a load of 98 kN was sustained. Nichols and Rapp (2000) comment that these results are very encouraging in terms of the durability of straw bale walls when multiple significant loads are applied, as may be the case in an earthquake scenario.

Ash et al. (2003) agree with Carrick and Glassford (1998) that render is key to improving shear resistance of straw bale walls. They investigated in plane cyclic loading of straw bale walls and tested six walls measuring 2.44 metres square. Three were earth rendered and three were cement rendered with three different types of reinforcement used with each render. The walls were two bales wide and were stacked using running bond. In the most heavily reinforced wall the bales were pinned to the sole plate using threaded bar. On four of the walls cross ties were inserted to tie the two render skins together in an attempt to prevent buckling of the render surface. An in plane horizontal load was applied to the top of the walls. The authors found that of the earth rendered walls there was a 25% increase in load at all displacements between the walls with the lightest and heaviest reinforcement. The cement rendered walls showed a similar pattern, however the difference between the lightest and heaviest reinforcement was over 100%. All of the cement rendered walls had a higher racking shear resistance than the earth rendered walls. Comparing all of the results the authors found that by changing the render type and the reinforcement in the render the racking shear resistance of the walls could be increased by almost six times. This work found a greater difference in performance of earth and cement renders than Faine and Zhang

(2002) found. Since the render has a large effect on the stiffness of straw bale walls, changing the type of render is likely to also have a significant effect on the overall stiffness of straw bale walls.

At the University of Bath Lawrence et al. (2009a) investigated the racking shear resistance of ModCell panels, but still limited to a cladding application. Tests were carried out on corner joints to investigate the resistance offered to racking shear. This work found that even with corner bracing the joints were not sufficient alone to resist the required racking forces. Lawrence et al. (2009a) then confirmed this by testing a two metre by two metre ModCell timber frame, timber frame with the straw bales and finally a fully rendered panel. The rendered panel was 3.5 times stiffer than the straw filled panel. Racking shear load resistance tests were then performed on full size ModCell panels measuring 3.08 metres wide, by 3.34 metres high, by 0.48 metres thick. Panels reinforced with steel bracing as well as unreinforced panels were tested. Cracks developed in the render of the reinforced panel at 1.25 times the load in the unreinforced panels and failure occurred at nearly three times the load. It was observed that load capacity and lateral stiffness of the frame is significantly influenced by the joint and reinforcement details. In addition Lawrence et al. (2009a) also observed that the render is a key contributor to the shear resistance of the panels. This observation is in keeping with other straw bale building techniques. Mesh reinforcement was not used in any of the render, but Lawrence et al. (2009a) point out that the vertical reinforcing bars have the same effect by adding some tensile strength to the render.

2.5 Conclusions

From this literature review it can be concluded that the straw and render in straw bale walls are key to structural strength. The importance of mesh reinforcement in the render is less clear as Ash et al. (2003) concluded that reinforcement improved the strength of straw bale walls where as Lawrence et al. (2009a) did not use mesh in any of their testing and yet still gain favourable results. Straw bale walls require the straw and the render to work compositely in order that maximum strength is achieved. Vardy and MacDougall (2004) found that render did not adhere to bales laid on edge as well as it did to bale laid flat. During testing the compressive strength of the bales laid on edge was 36% lower than those laid flat. They commented that some of the loss in strength was due to the render becoming separate from the straw at lower loads.

From the review of vertical compression testing it can be concluded that the following improve the strength and stiffness of straw bale walls;

- adding render
- pinning the bales together
- confining the render between timber sole and head plates
- using high density bales
- pre-compressing prior to rendering.

These are all currently considered to be good practice when constructing in straw bale and are all already implemented in the design of ModCell panels.

Zhang (2002) commented that initial displacements of 3 to 4% were observed when the bales were loaded. Faine and Zhang (2002), Walker (2004), Carrick and Glassford (1998) and Lawrence et al. (2009a) all pre-compressed some or all of their wall panels prior to rendering to prevent this and improve the stiffness of the walls. Walker (2004) tested both pre-compressed and un-compressed walls and found that the settlement under vertical load was reduced by 50% when the bales had been pre-compressed. As previously mentioned the bales in ModCell panels are pre-compressed by roughly 1.5% to reduce the risk of settlement once they are completed.

Finally as with the vertical strength, the racking strength of straw bale walls is largely affected by the render and its detailing. All of the research into in plane load resistance concluded that the contribution of the render is key to the strength and stiffness. Specifically looking at ModCell panels Lawrence et al. (2009a) showed that once render was added to a panel its stiffness increased by 3.5 times. However Lawrence et al. (2009a) comment that it is all elements in the panel working in composite that give the panel its racking shear resistance.

3 Laboratory Testing

3.1 Introduction

The laboratory testing in this thesis was carried out in the Department of Architecture and Civil Engineering at the University of Bath by the author. In order to achieve the aims of this investigation set out in Section 1.5 the testing outlined in Table 3.1 was carried out.

Table 3.1 Testing Methods and Information Required

	Testing Method	Information required
Materials	Lime render testing	The flexural and compressive strength of the render at the time of panel racking testing.
	Straw bale characterisation	The density and moisture content of the straw bales at time of construction.
Components	Joint testing – Vertical pull out	The vertical pull out strength and stiffness of the different joints.
	Joint testing – Rotational stiffness	The rotational stiffness of the joint selected for use in the full size panels for computer modelling of the panels.
Panels	Full scale panel racking shear testing	The racking strength of the panels, particularly at the maximum serviceability displacement. Displacement around the panel perimeter so the displacement of the panel can be mapped.
	Vertical compression test on panel	The failure load, location and loads at which cracking occurred, vertical displacement of panel.

3.2 Characteristics of materials

Details and properties of all of the materials used during this investigation are in shown in Table 3.2. All of the properties were derived from laboratory testing by the author unless otherwise stated.

Table 3.2 Material Properties

Material	Properties
Glue laminated timber	Supplied by Eurban Construction as strength class C24 timber Density: 450 kg/m^3 (All information below taken from BS EN 338:2003) Bending strength, $f_{m,k}$ = 24 N/mm^2 Tension parallel, $f_{t,0,k}$ = 14 N/mm^2 Tension perpendicular $f_{t,90,k}$ = 0.5 N/mm^2 Compression parallel $f_{c,0,k}$ = 21 N/mm^2 Compression perpendicular $f_{c,90,k}$ = 2.5 N/mm^2 Shear $f_{v,k}$ = 2.5 N/mm^2 Mean modulus of elasticity parallel $E_{0,mean}$ = 11 kN/mm^2 Mean modulus of elasticity perpendicular $E_{90,mean}$ = 0.37 kN/mm^2 Mean shear modulus G_{mean} = 0.69 kN/mm^2
Two string wheat straw bales	Sourced from Knowle Farm in Marlborough, Wiltshire Average density (at time of construction): 118 kg/m^3 Moisture content (at time of construction): 12 – 14%
Formulated lime render	Limetec Basecoat render Supplied by Lime Technology Ltd Mix (when sprayed) Water : Lime 1 : 4 (by mass) Flow table value: 148 mm 28 day compressive strength: 3.14 N/mm^2 28 day flexural strength: 1.33 N/mm^2
Spax Screws	8 mm diameter 200 mm long torx drive screws
Stainless Steel dowels & threaded bar	All information below taken from BS EN 1993-1-4:2006 Grade 304L (1.4307) Yield Strength, f_y= 200 N/mm^2 Ultimate Strength, f_u= 520 N/mm^2 Youngs Modulus, E= 200000 N/mm^2 Shear Modulus, G= 76923 N/mm^2

3.3 Lime render testing

There is a need to understand the long term performance of the render system used on the ModCell panels. The panels are delivered to site 14 days after rendering has been completed. Therefore, it is important to understand how strong the render is at this point and how this strength will subsequently develop.

During the construction of all the ModCell panels used for structural testing, specimens of the lime render were taken directly from the renderers spray machine. The specimens were 160mm long by 40mm wide by 40mm high. In total 18 specimens were made following the method set out in BS EN 1015-11:1999 and flow values of the render were measured according to BS EN 1015-3:1999. The specimens were cast in steel moulds and as a result they are unlikely to be a true representation of the render when applied to straw bales. The render on straw bales will dewater more rapidly due to absorption by the straw. However using steel moulds is the most consistent way of producing render specimens for the purpose of comparisons. These were then tested in flexure and compression using the Dartec 100 kN testing frame following the method specified in BS EN 1015-11:1999 at 7, 14, 28, 91 and 182 days (see Figure 3.1 and Figure 3.2). Three specimens were tested at each age giving three flexural strength results and six compressive strength results. The remaining three specimens were tested at the same time as the structural tests on the panels.

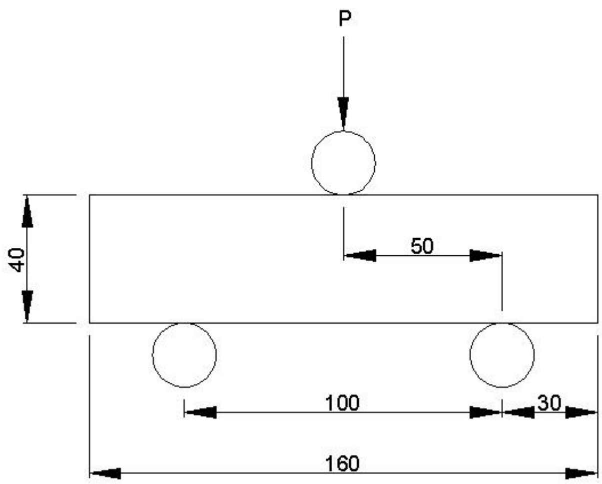

Figure 3.1 Render prism flexural test set up (BS EN 1015-11:1999)

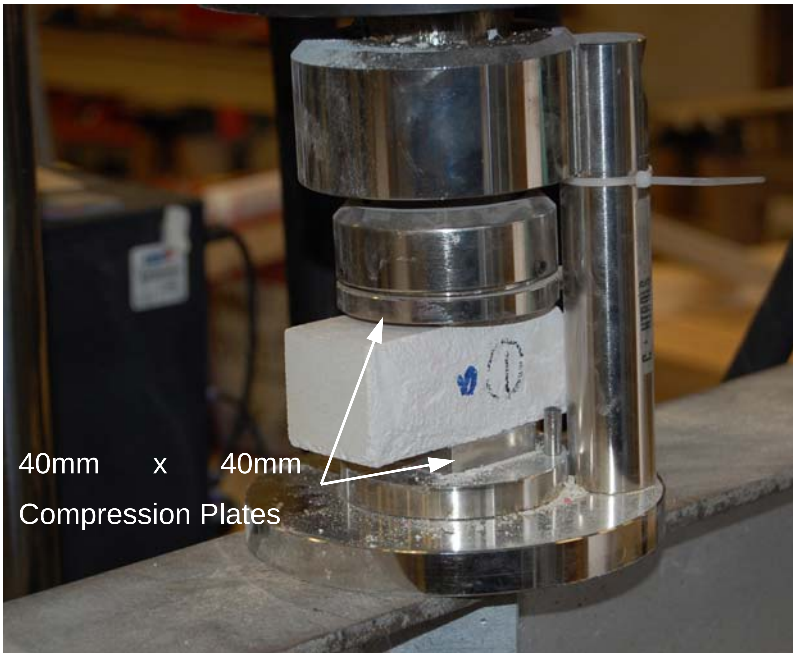

Figure 3.2 Render prism compression test set up

From the results the flexural and compressive strengths of the render can be calculated. The flexural strength is calculated from:

$$f_f = 1.5 \frac{P_f l}{bd^2} \qquad (3.1)$$

Where:

f_f Flexural strength,

P_f Load applied,

l Span of specimen,

b Width of specimen,

d Depth of specimen

The compressive strength is calculated from:

$$f_c = \frac{P_c}{40 \times 40} \qquad (3.2)$$

Where:

f_c Compressive Strength,

P_c Failure Load,

3.4 Joint testing

Joint testing was undertaken as previous research on ModCell panels (Lawrence et al. 2009a) had shown that the joints were a limiting factor in both the stiffness of ModCell panels and also the overall strength. When the horizontal in plane load is applied during full scale racking tests the top element of the timber frame has a tendency to move upwards. This puts a vertical load through the joints. In the previous work by Lawrence et al. (2009a) the joints failed by pulling apart vertically. Therefore it was decided that a series of joints should be tested in this manner to determine the most suitable to use in ModCell panels.

3.4.1 Vertical Pull out testing

The test set up for vertical pull out joint tests is shown in Figure 3.3. The test joints were constructed from one metre lengths of the glue laminated timber used in the ModCell panel frames. It should be noted that previous joint and panel tests undertaken by Lawrence et al. (2009a) used three ply cross laminated timber. Two pieces of glue laminated timber were used to make a joint at 90^{o}. One side of the joint was then fixed to the laboratory strong floor and the other piece was loaded vertically using a hydraulic jack and hand pump. Displacement was measured in four locations using Linear Voltage Displacement Transducers (LVDT) and logged using a System 6000 data scanner and computer. The joint displacement can be calculated using the following equation:

$$\delta = \frac{(LVDT3 - LVDT1) + (LVDT4 - LVDT2)}{2} \qquad (3.3)$$

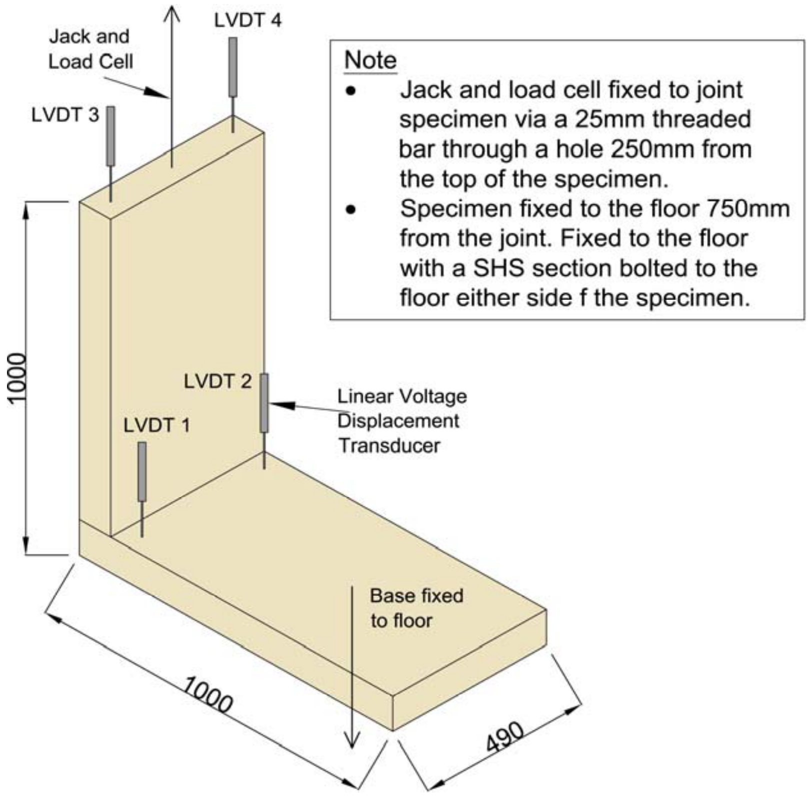

Figure 3.3 Vertical pull out test set up

3.4.2 Rotational Stiffness

The rotational stiffness of the joints was required for the development of a theoretical computer model of ModCell panels. Rotational joint tests were performed by opening and closing test joints from a 90^o starting position. Both opening and closing tests were performed as the stiffness may be different in each case and during racking shear loading of the panels two joints open and two joints close. The test set up is shown in Figure 3.4. The same size specimens were used as for the vertical pull out joint testing described in Section 3.4.1 of this report. The horizontal section of the joint was fixed to the laboratory floor to prevent sliding. The vertical section was loaded horizontally using a hydraulic jack and hand pump. Displacements were measured using LVDT and the load was measured using a load cell. The data was recorded on the System 6000 data scanner and computer. Any slip of the specimen across the floor is measured with LVDT 3. This is then subtracted from the displacement recorded on LVDT 1 and LVDT 2 to give the correct displacement.

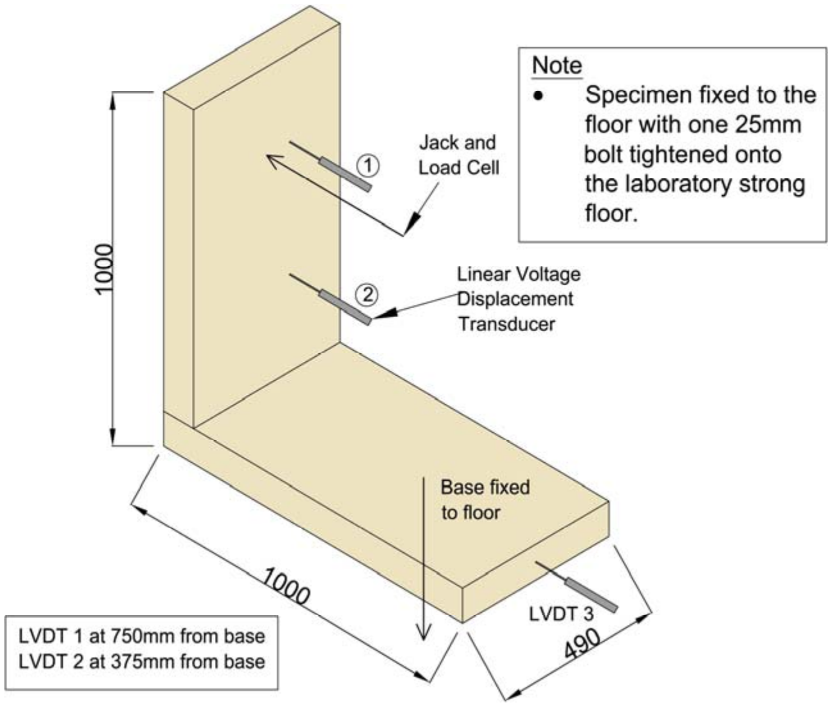

Figure 3.4 Rotation stiffness test set up

3.5 Full scale racking shear testing

Full size racking shear tests are designed to replicate the in plane loading conditions ModCell panels may be subject to in a building. Wind is the primary cause of in-plane loading; for a 6.8 metre by 6.8 metre house constructed from ModCell panels the total design wind load is in the order of 35 kN. This was calculated following the method set out in BS 6399-2 and the following parameters were assumed:

- Basic wind speed = 21m/s
- Site altitude 180m above sea level
- Located in the countryside
- 40km upwind from the sea
- South west wind direction

The set up for the tests are shown in Figure 3.5, Figure 3.6 and Figure 3.7. The ModCell panel is fixed to the laboratory floor to prevent sliding or uplift. A horizontal load is applied to the top element of the frame at the corner of the panel using a hydraulic jack and hand pump. Displacements are measured using LVDTs at set locations around the panel and the load was measured using a load cell. The data was recorded on a System 6000 data scanner and computer. A vertical surcharge load was not applied to the panels during testing as all vertical loads are transferred through the upright elements of the timber frame. While a surcharge load may affect the racking shear resistance, it is felt that this affect is small because vertical loads pass through the vertical frame elements and therefore can be ignored during this investigation.

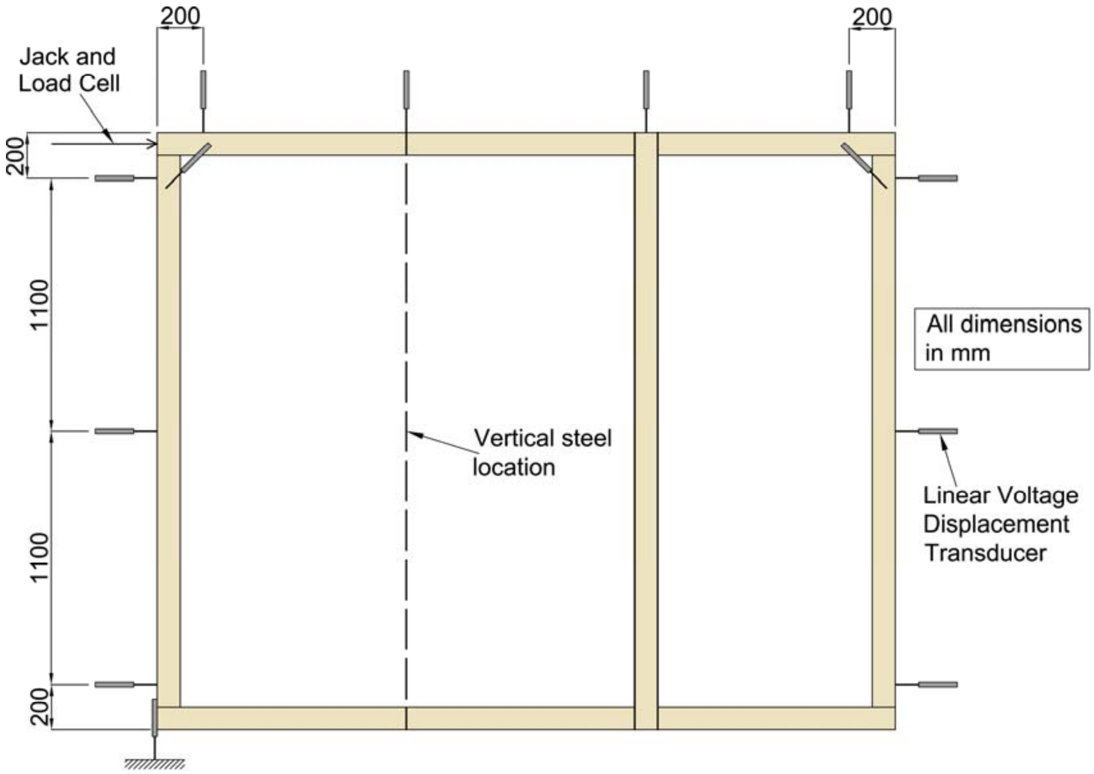

Figure 3.5 Two bale panel racking test set up

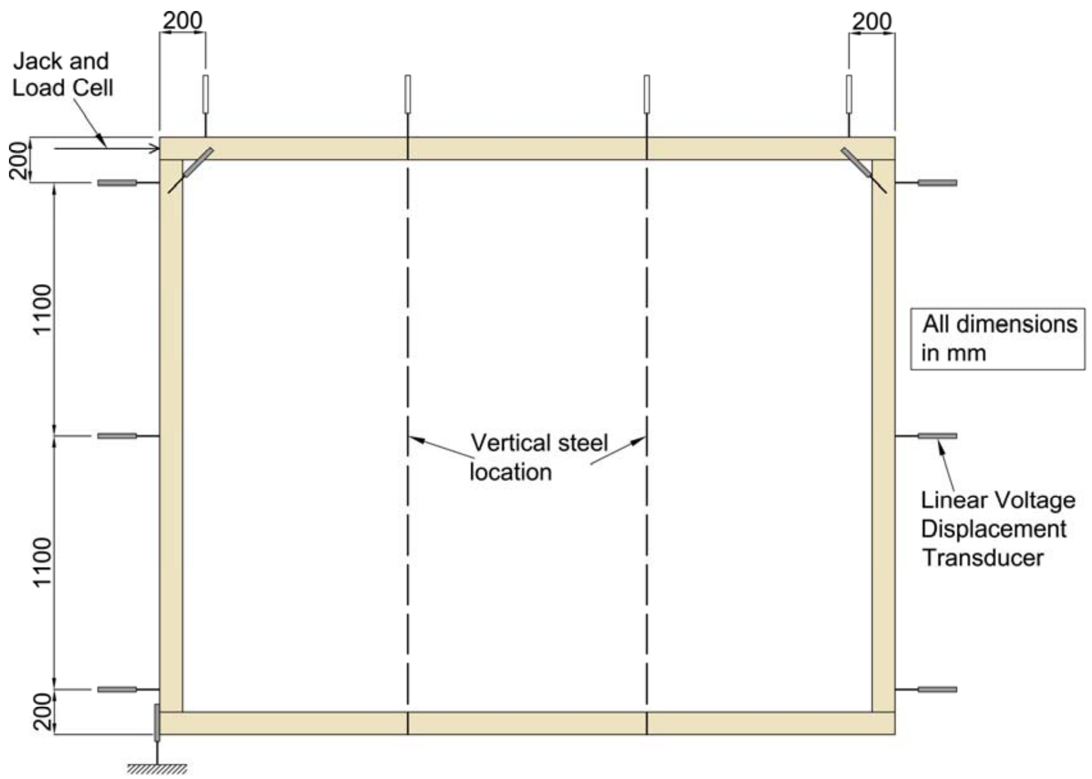

Figure 3.6 Three bale panel racking test set up

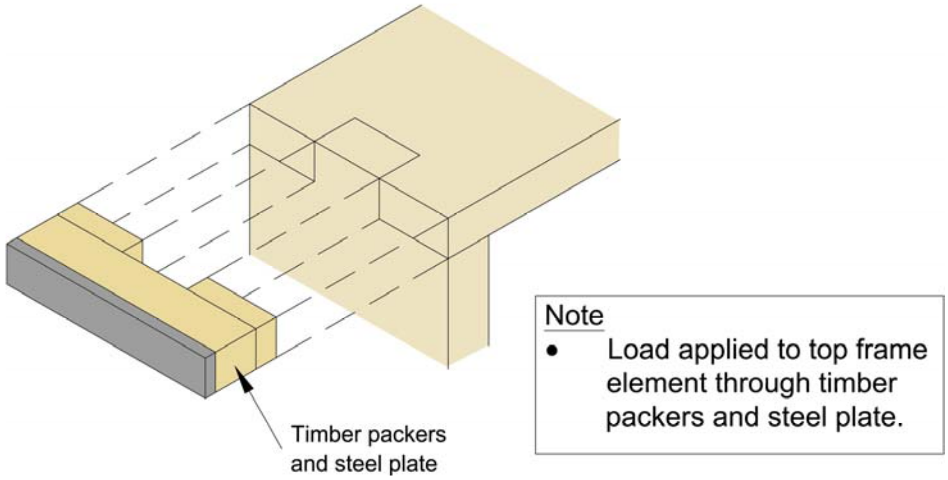

Figure 3.7 Load application during racking testing

3.6 Vertical compression testing

Vertical compression testing was carried out in order to assess the level of vertical load that could safely be placed on top of ModCell panels. One can assess the level of load the top element of the timber frame can safely carry by means of a simple calculation, but working out the amount of load the straw and render can carry is more complex. This needs to be established as when the timber is loaded it will deflect and as a result load the straw and render. Under this loading the render cannot crack, fail or deform excessively.

The test set up is shown in Figure 3.8. An existing 1.9m by 1.2m wide ModCell panel was used for this test. The panel had been used for exposure testing and had been exposed to the weather for a year prior to the vertical compression test. The timber frame was constructed from three ply cross laminated timber. The top element of the timber frame was cut at its joints to allow it to move freely downward when loaded without any interference from the sides of the timber frame. The sides of the frame are constrained to stop them rotating outward when load is applied to the top of the panel. A vertical load is applied to the top element of the frame via a spreader beam using a hydraulic jack and hand pump. The displacement of the top of the panel was measured at its four corners using LVDT and the load was measured using a load cell. The data was recorded on a System 6000 data scanner and computer.

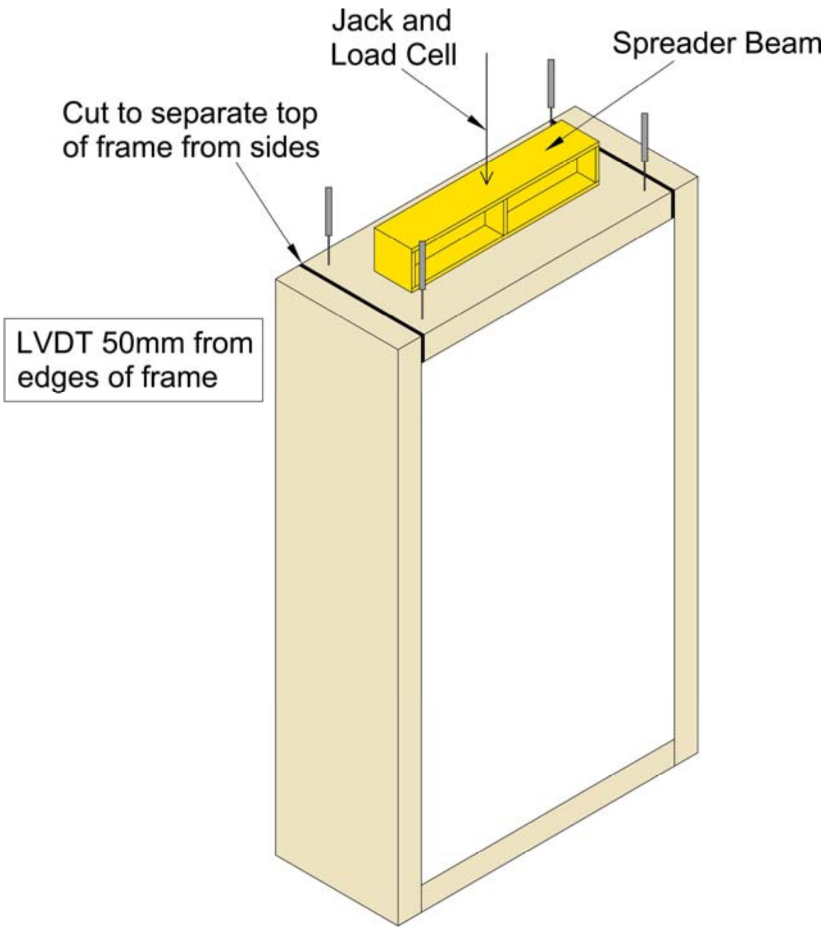

Figure 3.8 Vertical compression rig with ModCell panel

3.7 Straw bale Characterisation

Throughout the construction of the ModCell panels used for racking shear testing a sample of the straw bales used was taken. These were tested for moisture content using a Balemaster probe. The average moisture content at the time of construction was 13.5 %H_2O. The average density of the bale prior to trimming was calculated by taking a sample of six bales and measuring their weight and volume. The average density was 118.4 kg/m^3. The full results from these tests are shown in Table 3.3. The straw bale moisture contents are the levels that would normally be expected in bales that have been stored under cover and protected from the weather. Straw bales with a moisture content of between 10 and 15 %H_2O are suitable for the construction of the panels. There is no significant variation in the moisture contents, weights or volumes which shows that all of the bales to be used in the construction of the panels are likely to be a similar quality.

Figure 3.9 Balemaster probe

Table 3.3 Survey of Bales used in construction of panels

Bale	Volume (m^3)	Weight (kg)	Moisture Content (% H_2O)
1	0.154	17.6	12.1
2	0.159	18.8	13.4
3	0.163	19.3	13.8
4	0.169	20.1	14.1
5	0.164	19.9	13.6
6	0.170	20.2	14.0
Average	0.163	19.3	13.5

4 Material and Component Properties

4.1 Properties of Render

Render specimens were tested in both flexure and compression at 7, 14, 28, 91, and 182 days. All of the specimens were made from the same sample of render taken directly from the renderer's spraying machine. At the time of making the specimens a flow table value of 148 mm was recorded in accordance with BS EN 1015-3:1999.

Details of the specimens tested are shown in Table 4.1. All of the flexural strength tests were carried out at a constant displacement rate of 0.3 mm/min. This displacement rate ensured failure between 30 and 90 seconds as set out in BS EN 1015-11:1999. The compressive strength tests were carried out with a constant displacement rate of 0.5 mm/min.

The flexural and compressive results for all the specimens tested are shown in Table 4.1. From the results both the flexural and compressive average strengths were calculated for each specimen age. These are displayed in Figure 4.1 where age is plotted against strength. The coefficient of variation for each age was also calculated and is shown in Table 4.1.

Table 4.1 Long term lime render test specimens details and results

Specimen	Age (days)	Cross Section b (mm)	Cross Section d (mm)	Density (kg/m^3)	Flexural strength (N/mm^2) Test 1	Flexural strength (N/mm^2) Average	Coefficient of variation (%)	Compressive strength (N/mm^2) Test 1	Compressive strength (N/mm^2) Test 2	Compressive strength (N/mm^2) Average	Coefficient of variation (%)
1	7	41	40	1761	0.72	0.69	6.09	1.34	1.35	1.33	3.04
2	7	41	40	1753	0.63			1.31	1.28		
3	7	41	40	1730	0.73			1.31	1.41		
4	14	40	40	1555	1.36	1.26	6.02	2.84	3.02	2.90	3.22
5	14	40	40	1566	1.24			2.87	2.99		
6	14	40	40	1551	1.17			2.75	2.90		
7	28	42	40	1488	1.33	1.33	7.12	3.18	3.06	3.14	2.21
8	28	41	40	1540	1.45			3.19	3.21		
9	28	41	40	1513	1.22			3.18	3.03		
10	91	42	40	1522	1.17	1.19	2.97	2.79	2.78	2.93	3.82
11	91	41	40	1536	1.23			2.97	2.97		
12	91	42	40	1496	1.15			3.10	2.97		
13	182	42	40	1510	1.17	1.18	1.43	2.58	2.60	2.54	3.07
14	182	42	40	1518	1.18			2.43	1.99*		
15	182	43	40	1486	1.19			2.46	2.62		

* Discounted as strength considerably lower than other specimens

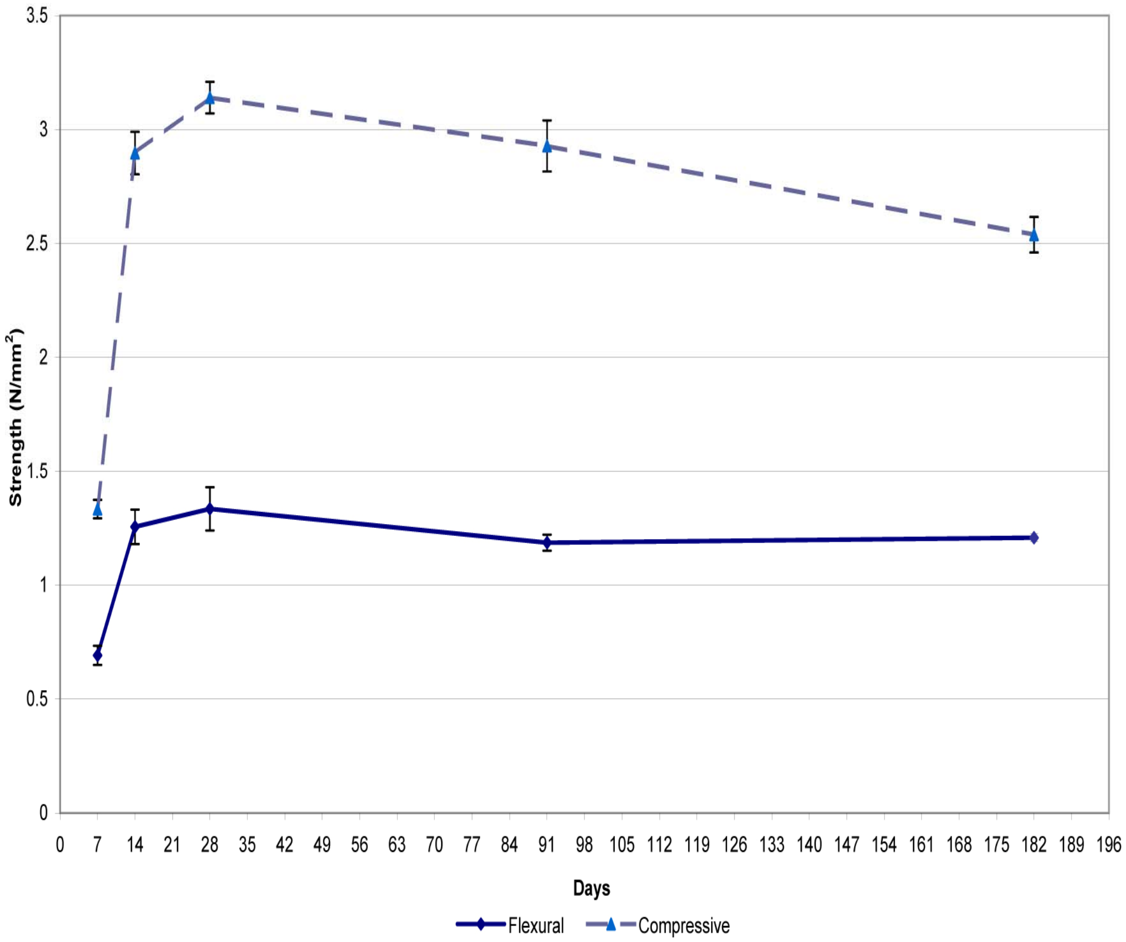

Figure 4.1 Lime render strength

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In both flexure and compression at seven days the render has roughly 50% of its final strength (Figure 4.1). This is a very quick strength gain. In comparison a 1:1.5 NHL 3.5 mortar has 7% of its final compressive strength and 13% of its final flexural strength at seven days (Allen et al. 2003). At 14 days the render has gained both its full flexural and compressive strength. This is important as the completed panels are generally transported to site 14 days after rendering has taken place. These results have confirmed that there is little benefit in leaving the panels to cure for longer than 14 days before transportation. The results also indicate that the Limetec Basecoat render system being used is particularly suitable for this application as it gains strength very quickly when compared to traditional lime renders. The fast gains in strength are due to the cementitious constituents in the formulated render mix.

Both the flexural and compressive results show a drop-off in strength after 28 days. For the flexural strength this drop-off levels at around 91 days whilst the compressive strength drop-off appears to continue (Figure 4.1). The drop off in strength both in flexure and compression is currently unexplained and requires further investigation. One possible for it could be the development of micro cracks as the render is slowly curing which will cause weaknesses when tested. The specimens were cured in their moulds under sealed polythene for seven days and before being de-moulded and transferred to a conditioning room with a constant temperature of 21^{o}C and relative humidity of 65%. As long as the renders strength levels off to acceptable values the drop off in strength is not a problem.

Figure 4.1 shows error bars as a standard deviation percentage for all the specimens tested. Generally as the age of the specimens increases the error reduces. This shows that they may initially be curing at slightly different rates, which could be due to the position the specimens were within the moulds or the conditioning room. The specimens closest to the sides of the polythene may have cured more rapidly due to leakage of air around the edges.

As the age of the render increased the amount of carbonation also increased. This was tested following flexural testing. The freshly broken surface of the render was sprayed with phenolphthalein solution. Figure 4.2 to Figure 4.6 show the amount of carbonation for each specimen age. Note the solution turns pink on areas that have not carbonated. At seven days there was no carbonation as the specimens were removed from their moulds at five days and kept in sealed plastic bags until seven days and hence they were not exposed to the air. At 182 days the render specimens are almost fully carbonated with only a faint pink area in the centre. These results indicate that the 30 mm thick render on ModCell panels would take roughly 150 days to fully carbonate.

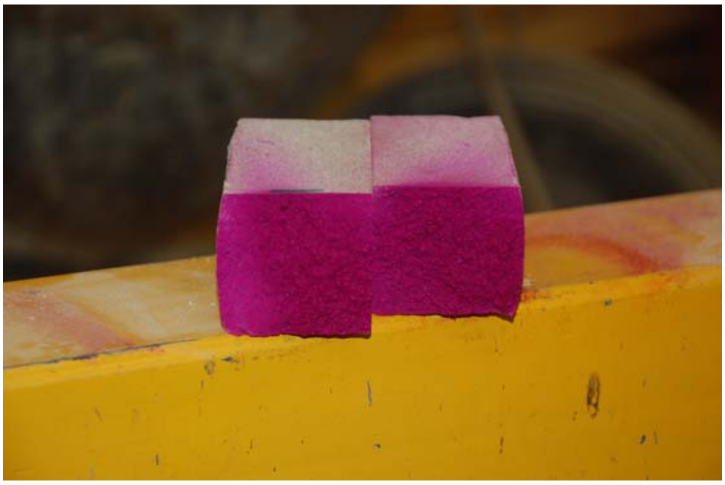

Figure 4.2 Carbonation of 7 day render specimen

Figure 4.3 Carbonation of 14 day render specimen

Figure 4.4 Carbonation of 28 day render specimen

Figure 4.5 Carbonation of 91 day render specimen

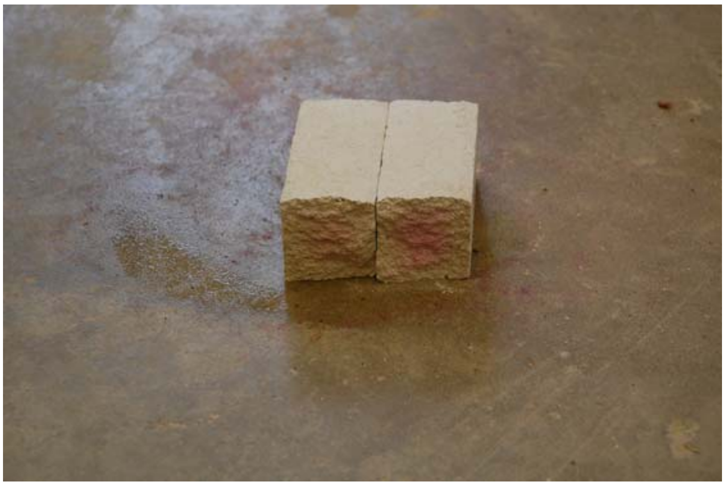

Figure 4.6 Carbonation of 182 day render specimen

4.2 Performance of Joints in Combined Shear and Tension

Joint testing was carried out for the reasons described in Section 3.4 of this report. All of the specimens were constructed and tested in the Structures Laboratory at the University of Bath by the author. The results of these tests are presented and discussed in the following sections of this report.

4.2.1 Joint Details

Three different designs of joint were subjected to vertical pull out test in order to determine which was the most suitable for use in ModCell panels. The three types of joint tested were: screw connected; dowel connected; and dovetailed. Details of these joints are shown in Figure 4.7, Figure 4.8 and Table 4.2. The one metre lengths of timber used to construct the joints were double ended to allow them to be used for two different joint tests. There were four complete specimens for each type of joint, so a maximum of eight tests could be undertaken for each type of joint. The screw connected joint was tested with eight screws being loaded axially and four screws being loaded in shear.

Table 4.2 Joint connection details

Joint type	Connection details
Screwed	Twelve 8mm diameter 200mm long countersunk head screws. 6mm diameter pilot hole through the first element. Joint pulled in tight with screws.
Dowelled	10mm diameter stainless steel dowel in a 10mm diameter pre-drilled hole. Joint pushed together and dowel hammered through.
Dovetail	Pre-cut dovetail push fitted together.

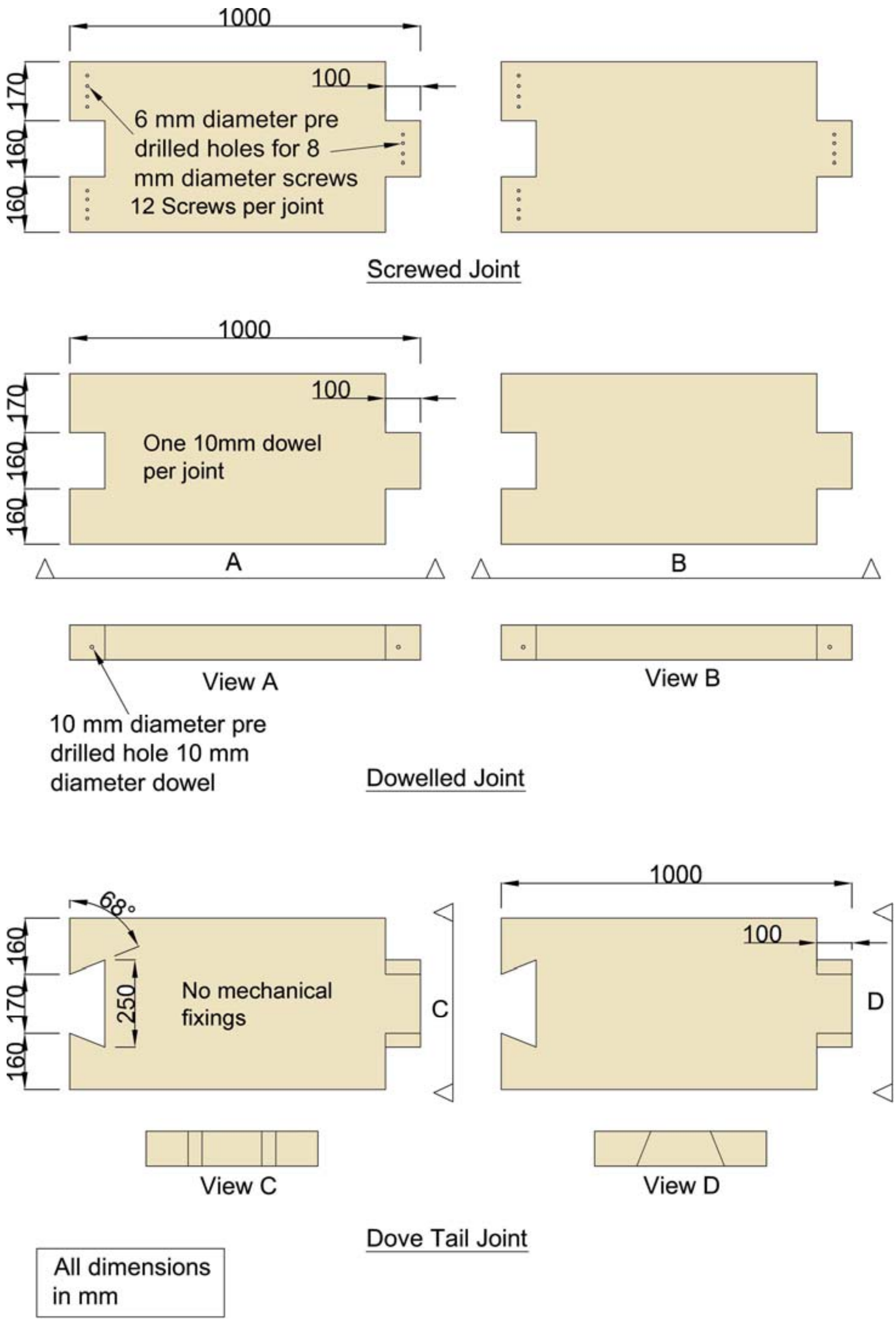

Figure 4.7 Details of joint types

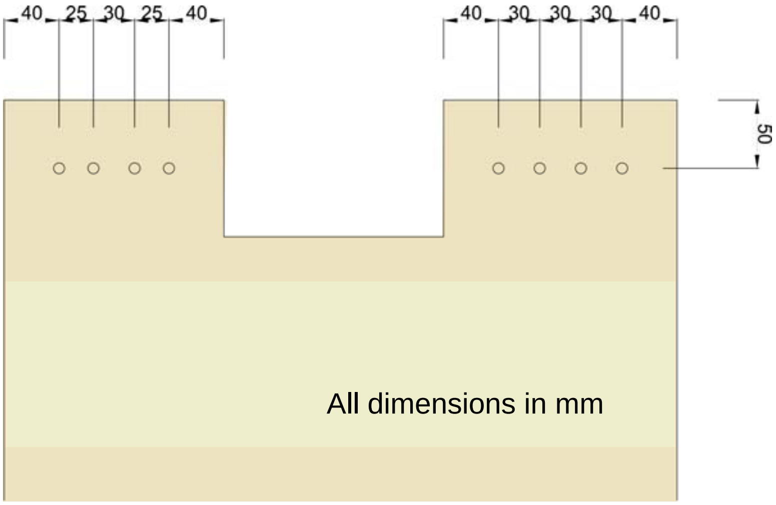

Figure 4.8 Screw edge distances

4.2.2 Vertical pull out

Figure 4.9 shows the results from the vertical pull out testing. Figure 4.10 shows average results from the joint pull out tests. Both graphs plot load (kN) against average vertical displacement (mm). Table 4.3 shows maximum load, stiffness and coefficient of variation for the joints. It should be noted that the stiffness for the joints was taken at 50% of peak load to remove the effects of any initial ‘play’ in the joints. This is particularly important in the dovetail joints. 50% has been used as it is at this point that the dovetail joints have fully tightened and are displaying their full stiffness.

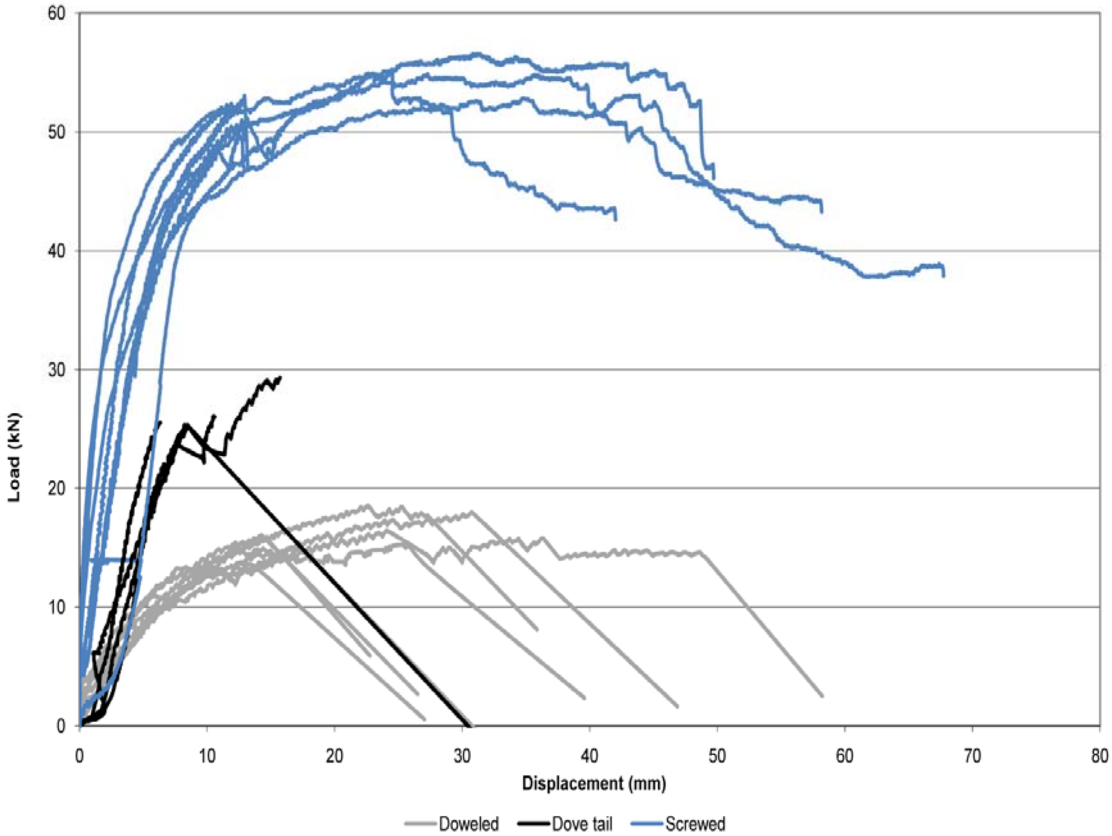

Figure 4.9 Vertical pull out test results

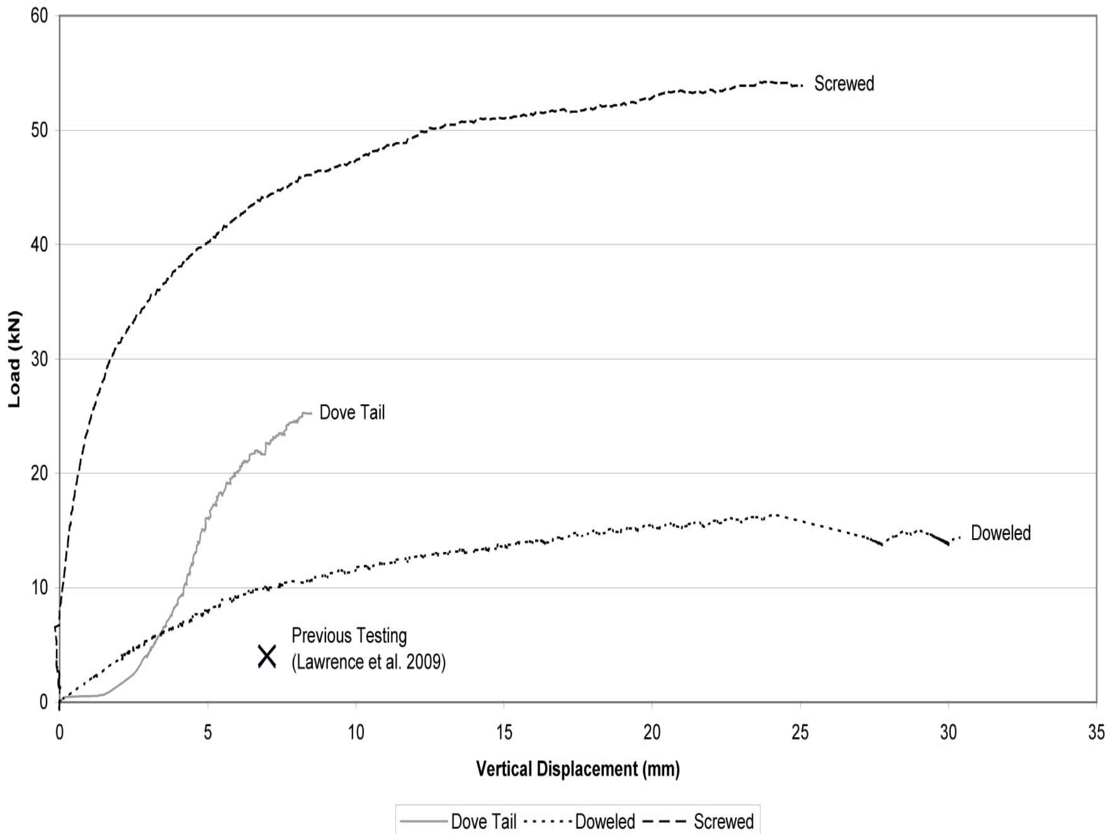

Figure 4.10 Typical behaviour of joints in vertical pull out tests

Table 4.3 Results from joint testing

Joint	Max Load (kN)	Coefficient of variation for max load (%)	Stiffness (kN/mm)	Coefficient of variation for max stiffness (%)
Screwed No. 1	56.53	3.78	10.21	25.01
Screwed No. 2	55.21		11.73	
Screwed No. 3	54.86		9.27	
Screwed No. 4	52.82		5.67	
Screwed No. 5	52.96		7.16	
Screwed No. 6	51.19		6.42	
Screwed No. 7	52.36		9.99	
Screwed No. 8	50.11		6.32	
Doweled No. 1	13.63	9.16	2.13	26.95
Doweled No. 2	17.93		1.00	
Doweled No. 3	16.41		1.07	
Doweled No. 4	15.24		1.40	
Doweled No. 5	15.72		1.20	
Doweled No. 6	16.02		1.45	
Doweled No. 7	15.12		1.66	
Doweled No. 8	18.53		0.98	
Dovetail No. 1	25.27	6.20	6.62	23.95
Dovetail No. 2	29.34		3.91	
Dovetail No. 3	26.05		2.61	
Dovetail No. 4	25.48		6.12	
Previous	12.00		N/A	NA

During testing the three different types of joint behaved quite differently. Initially as the screw connected joints were loaded they exhibited a very high stiffness of between 6 kN/mm and 12 kN/mm. As the load was increased the heads of the screws in tension started to pull through the timber (Figure 4.11). Under increasing load the screws in tension continued to pull through and the screws in shear started to split the timber (Figure 4.12). The screw connected joint failed in a ductile manner. This is shown in Figure 4.10 as the line for the screw connected joint asymptotes to horizontal at around 54 kN. Although having a ductile joint is not essential it is a preferred quality as it gives some warning that there is a problem in the structure rather than a catastrophic failure occurring.

Figure 4.11 Screws pulling through during testing

Figure 4.12 Screwed joint showing splitting of the timber following testing

The dowel connected joints were the weakest and had the lowest stiffness. Initially as the joints were loaded the stainless steel dowel deformed (Figure 4.13). The joints eventually failed in a brittle manner with the timber on the horizontal portion of the joint splitting at the dowel location (Figure 4.14).

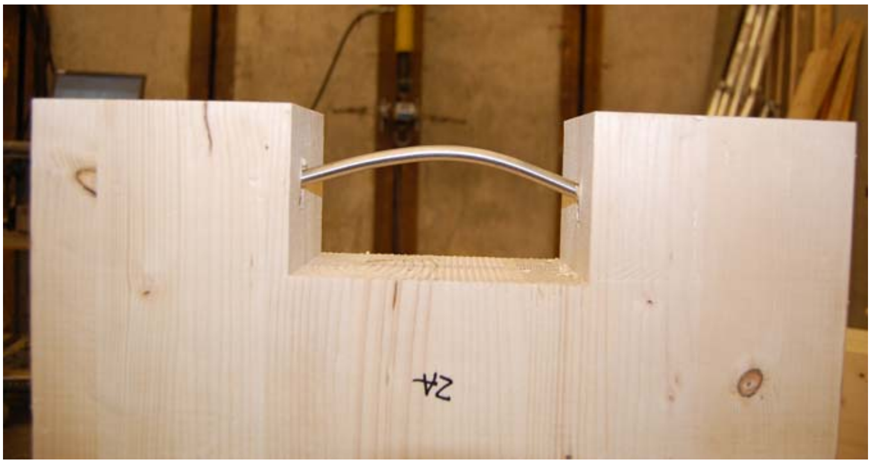

Figure 4.13 Deformed stainless steel dowel following testing

Figure 4.14 Timber splitting during testing (Dowel connected joint)

As previously mentioned the dovetailed joints were a push fit and even though they were CNC cut by the manufacturer to tolerances of ± 1 mm there was still some initial 'play' in the joint. Consequently there was roughly 2 mm of vertical displacement before the joint started to load. Once the joint started to carry load it was relatively stiff, however the joints failed in a brittle manner with the vertical section of timber splitting, sometimes explosively, along its length from top to bottom (Figure 4.15). This is not a desirable failure mode. As the testing completely failed the vertical sections of the joints they could not be used again and therefore only four dovetail joints were tested.

Figure 4.15 Timber failure during testing (Dovetail joint)

The average maximum load and displacement of a previous design of ModCell joint tested by Lawrence et al (2009a) is shown in Figure 4.10. These joints failed at an average load of 12 kN. This is considerably lower than any of the joints tested during this investigation therefore all of the new joints are an improvement over the previous design.

The aim of the joint testing was to find the joint that was the most suitable for ModCell construction. The joint needs to be both stiff and strong. Of the three types of joint tested the screw connected joint is both the stiffest and strongest. It is therefore the most suitable as the aim is to increase the stiffness of ModCell panels significantly so they can be used in load-bearing construction. As previously mentioned the screw connected joint has some ductility when failing which is desirable. From Table 4.3 it can be seen that the screw connected joints were the most consistent in terms of maximum load with a coefficient of variation of 3.78%. This is desirable in a building as it reduces the risk associated with the connection and the factors of safety used in design can be reduced.

There are also other reasons why the screw connected joint is the most suitable. Firstly ModCell panels are currently constructed using screw connected joints. Therefore this new improved joint is a logical evolution from the previous joint design. Screw connected joints are also familiar to the ModCell panel fabricators and as a result this is less likely to cause confusion or result in problems during the fabrication process. Finally, of the three joint designs tested the screw connected joint is the simplest and easiest to construct.

It can be concluded that the screw connected joint is the most suitable and it will therefore be used in the construction of full size panels for racking shear tests in this investigation.

4.2.3 Additional joint testing

Once the screw connected joint was chosen it was subjected to further vertical pull out testing in order to establish the contribution of each set of screws (screws in tension and screws in shear) to the strength of the joint. Joints were tested with eight screws in shear, four screws in shear, eight screws in tension and finally four screws in tension. These joints were tested using the procedure in Section 3.4.1 but the Dartec 2000 kN testing frame was used (Figure 4.16) instead of a hydraulic jack and hand pump as the testing was starting to damage the laboratory equipment. The average results from these tests are shown in Figure 4.17 which plots load against vertical displacement.

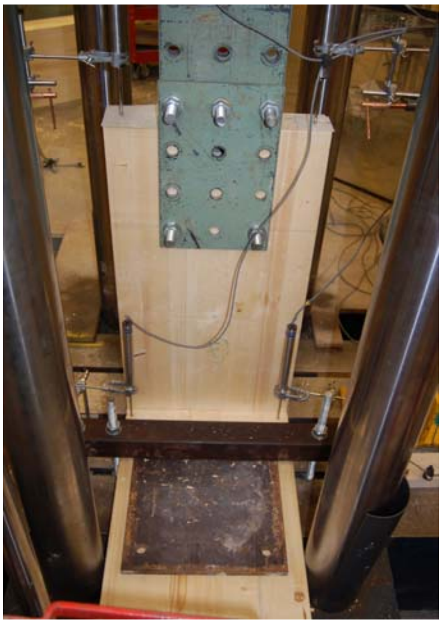

Figure 4.16 Joint testing in Dartec 2000 kN loading frame

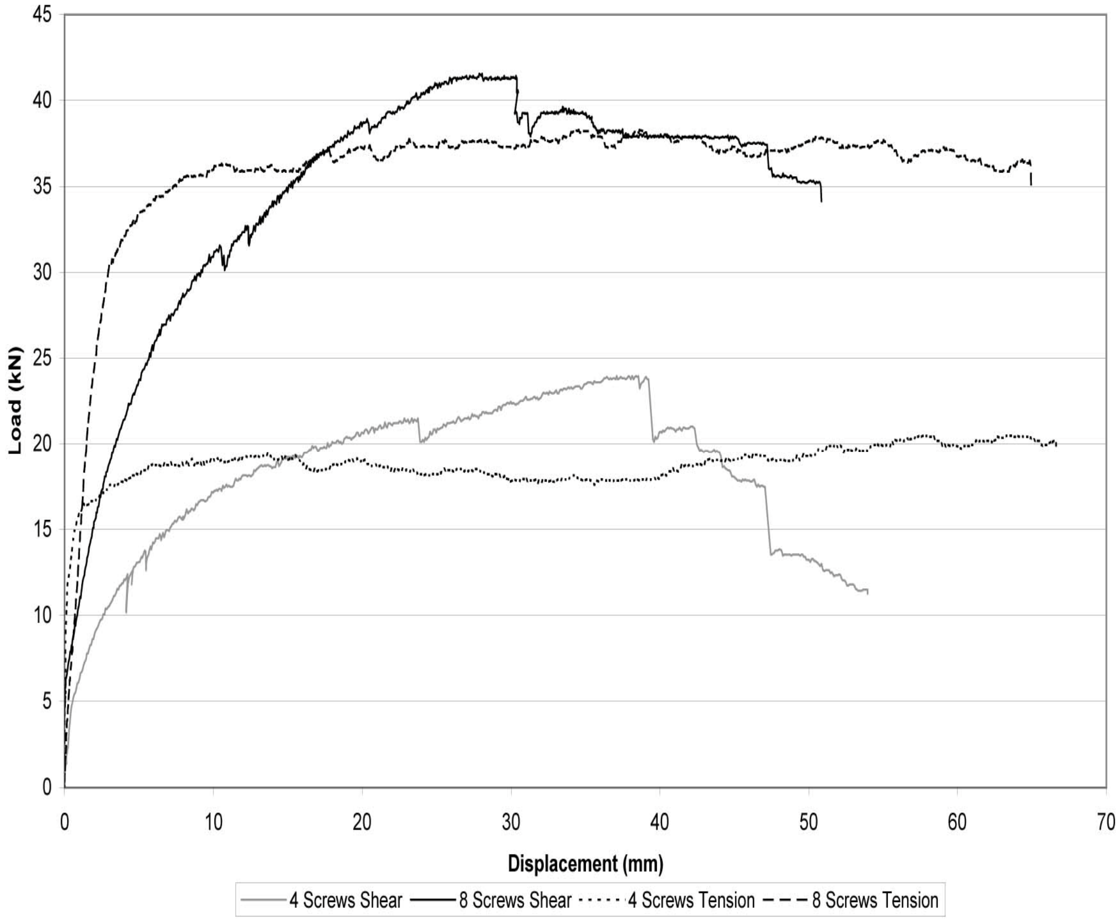

Figure 4.17 Results for joints with screws in either shear or tension

All of the joints with screws in tension failed as a result of the screw heads pulling through the timber (Figure 4.18). All of the joints in which the screws were loaded in shear failed by the surrounding timber splitting (Figure 4.19). This can be seen in Figure 4.17 where the load suddenly drops off for both the joints with four screws and eight screws. Figure 4.17 shows that the peak loads for both joints with screws in tension and joints with screws in shear were similar. This is a surprising result as during the testing of complete joints (see Section 4.2.2) with both shear and tension screws in the joint the screws in tension showed signs of failure at a much lower load than the screws in shear. The joints with eight screws were roughly twice as strong as the joints with four screws. This is to be expected as the screws were widely enough spaced so as not to greatly influence one another.

Figure 4.18 Tested joint showing tension screw pull through

Figure 4.19 Tested joint showing timber splitting

The joints with screws in tension were initially much stiffer than the joints with screws in shear. This is as a result of the higher resistance generated by the screw head bearing onto the timber. Thereafter, the load required draw the screw head through the timber was constant. The initial stiffness of the joint with screws in shear is provided solely by the screws as they deform.

From the results shown in Figure 4.17 it is possible to calculate the average maximum load a single screw can take in this type of joint in either tension or shear. These loads are shown in Table 4.4. The values for load per screw could be used to predict the performance of similar joints with different numbers of screws.

Table 4.4 Maximum load on a single screw

Joint	Total Joint load kN	Load per screw kN
4 Shear	23.96	5.99
8 Shear	41.56	5.19
4 Tension	20.54	5.14
8 Tension	38.27	4.78

This testing has shown that both the screws in tension and shear make an equal contribution to the joint strength. It also shows that if the joints were constructed with the same total number of screws, but with different numbers in shear or tension then the joints would perform in a similar way. However as the number of screws in shear is increased the initial stiffness of the joint may reduce as the screws in shear are initially less stiff. The spacing and edge distances of the screws will have had minimal affect on the performance of the joints as they are within the guidelines set out in Eurocode 5 (BS EN 1995 1-1:2004). Additionally all of the test specimens had the same edge distances and screw spacing regardless of the total number of screws in the joint.

4.2.4 Comparisons with Eurocode 5

The test results for the screwed joint were compared with calculated strengths using Eurocode 5 (BS EN 1995 1-1:2004). The average maximum experimental load recorded for the screwed joints was 54.85 kN (calculated from results in Table 4.3).

In order to calculate the strength some assumptions have been made as Eurocode 5 does not specifically cover axially loaded screws in the end grain of timber. Eurocode 5 states in clause 8.7.2 that one of the failure modes for axially loaded screws is head pull through, other failure modes include snapping of the screw head and pull out of the screw thread. From the joint testing in Sections 4.2.2 and 4.2.3 it is known that the heads of the axially loaded screws pull through when loaded. Eurocode 5 Clause 8.7.2 Note 6 states that the screw head pull through strength should be determined by laboratory testing.

Pull through testing was conducted on individual screws through a 210mm wide by 150mm deep by 100mm thick sample of the timber used for ModCell frames. Each specimen of timber had a 6mm diameter pre-drilled hole into which the screw was inserted (Figure 4.20). The timber sample was then inserted into a steel testing rig with the point side of the screw pointing vertically upwards from it. The timber was held in place and the screw was loaded using the 100 kN Dartec testing frame. Load and displacement were recorded.

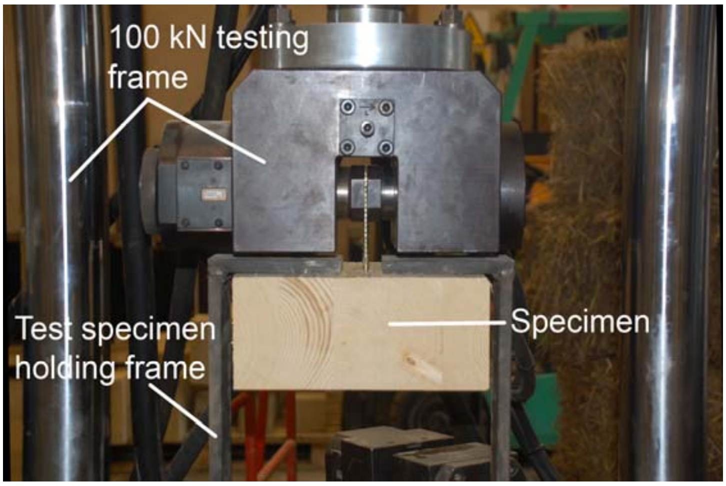

Figure 4.20 Screw pull through testing

The results are shown in Figure 4.21 and from these the average pull through capacity was found to be 5.0kN. This result has been used to find the calculated strength of the joints.

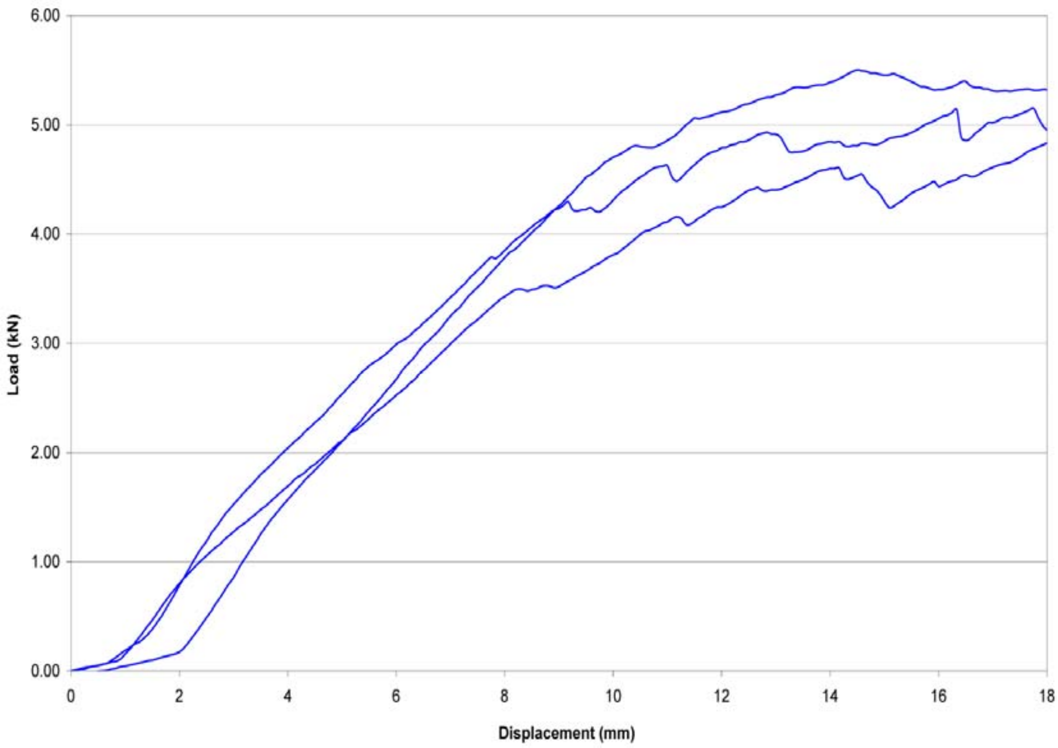

Figure 4.21 Average results from screw pull through testing

The average pull through strength was then used in the calculation of joint strength following Eurocode 5. This is shown below:

Joint with 12 screws. Four laterally loaded, eight axially loaded. All screws are 200 mm long with thread diameter of 8 mm.

Screw smooth shank diameter is 5.7 mm, $\therefore$ clause 8.7.1 (5) applies.

For laterally loaded screws:

From equations in clause 8.2.2, equation 8.6d gives the lowest value of Fv,Rk

$F_{v,Rk} = 5016.1N$ per fastener

Laterally loaded screws are fixed into end grain $\therefore$ clause 8.3.1.2 (4) can be applied

$\therefore$ Total load capacity of laterally loaded screws $= \frac{1}{3} \times 4 \times 5016.1 = 6688.1N$

For axially loaded screws:

Following clause 8.7.2, the average screw head pull through load is 5.00kN

$\therefore$ Total load capacity of axially loaded screws $= 5000 \times 8 = 40000N$

$\therefore$ Total Joint capacity $= 6688.1 + 40000 = 46688N = 46.69kN$

Therefore the tested joint strength was 54.85 kN and the calculated joint strength was 46.69 kN. This result shows that it is possible to calculate the strength of the screw connected joint using Eurocode 5. This will prove useful in the further development and design of load-bearing ModCell panels as the vertical pull out strength of the joints can be predicted using a standard and widely used design code. A comparison with BS 5268 has not been made as it does not allow for any fixings in the end grain of timber.

4.3 Rotational stiffness testing

The rotational stiffness of the screwed joints was derived by testing joints following the method outlined in Section 3.4.2. These data are required for development of a computer model of the panels. The joints tested used twelve 200 mm long screws as per the vertical pull out tests. The screws were arranged in an offset pattern (Figure 4.22 and Figure 4.23) as, due to an error during manufacture, this is how the full size ModCell panels tested during this investigation were constructed. The results from the full panel structural testing and the computer model were to be compared and therefore the constructions of both needed to be the same.

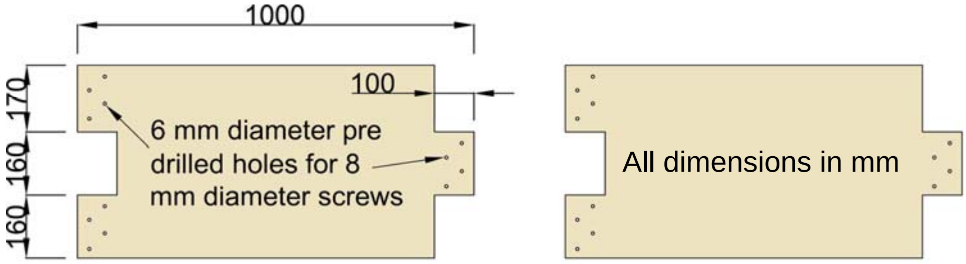

Figure 4.22 Screw layout for rotational stiffness testing

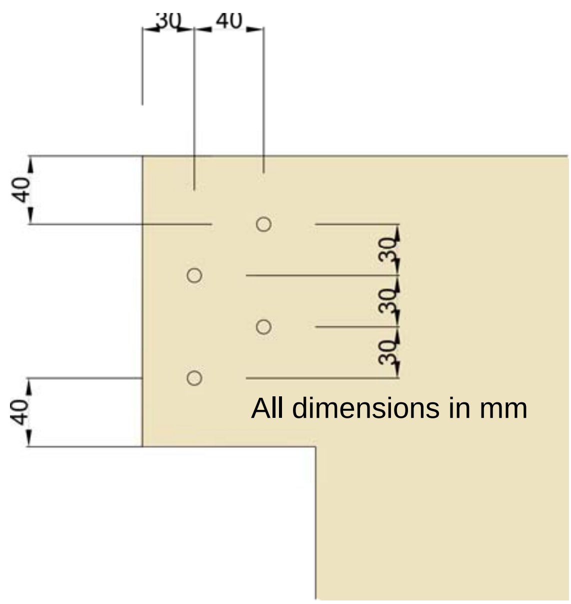

Figure 4.23 Screw edge distances

During testing of the joints in opening initially the screw heads started to pull through the timber. As the load was increased to roughly 2.67 kN (2kNm) the timber on the horizontal section of the joint split (Figure 4.24). Thereafter the load being carried by the joint remained constant. During testing of the joints in closing under load the heads of the screws started to pull through the timber (Figure 4.25). As the load was increased the timber did not split and as a result the load did not level off as quickly as with the joints in opening. The results from these tests are shown in Figure 4.26.

Figure 4.24 Joint in opening showing splitting of timber

Figure 4.25 Joint in closing showing screw head pull through

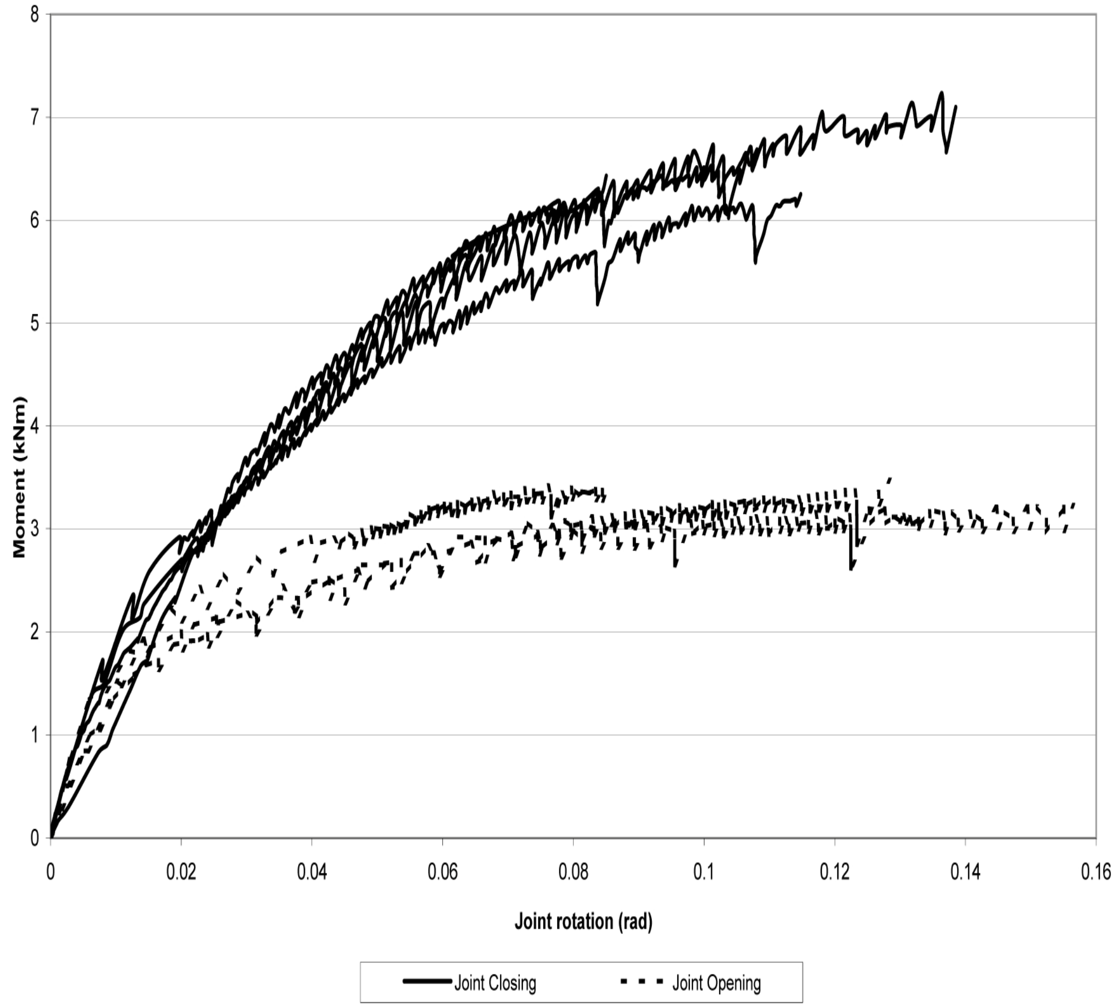

Figure 4.26 Joint testing results - Rotation

Within the limits of joint rotation critical to ModCell panel performance, both the opening and closing joints have similar initial stiffness. A joint rotation of 0.02 radians is equal to a horizontal displacement at the top corner of the panel of 52 mm. This is ten times the serviceability horizontal displacement limit of 5.2 mm (h/500). Therefore for the purposes of building a computer model the joints will be considered to have equal stiffness in both opening and closing. The average rotational stiffness of the joints in both opening and closing is 135 kNm/rad.

4.4 Conclusions

Long term testing of the lime render used on ModCell panels has shown that it:

- gains strength quickly when compared to other lime mortars

- the strength reduces after 28 days and the flexural strength levels off at 91 days but the compressive strength continues to drop
- the strength drops may be due to micro cracks and further investigation is required.

From the first set of joint tests, the following conclusions have been drawn about the screw connected joint:

- Strongest
- Stiffest
- Easiest to construct

The further testing on joints with screws in only one direction (either in tension or shear) has shown that both sets of screws make an equal contribution to the strength of the joints. It has also been shown that EC5 can be used to correctly calculate the design strengths of this type of joint which will help greatly in the design of ModCell panels.

Finally the rotational stiffness of the screw connected joint used ModCell panels for full scale racking shear testing has been shown to be 135 kNm/rad. These tests have also shown that within the limits of joint rotation critical to the performance of ModCell panels the stiffness of the joints in both opening and closing is the same.

5 Racking Shear Tests on Panels

5.1 Introduction

During racking shear testing two sets of panels were tested. Initially two panels from Grand Designs Live were tested. Following this four laboratory built panels were tested. This chapter examines both of these set of tests.

5.2 Grand Designs Panels

Two ModCell panels from “The House That Kevin Built” (part of the Grand Designs Live project at Excel in May 2008) were tested in racking shear. The house was the first time that load-bearing ModCell panels had been used. The panels were constructed in a temporary factory close to the house site by Agrifibre Technologies before being delivered to site. The house was constructed between 4th May 2008 and 9th May 2008 using ModCell for the ground floor with other building technologies making up the rest of the house. Once completed, the house remained onsite for a week before being dismantled and put into storage. Following this two of the panels were delivered to The University of Bath Structures Laboratory on 2nd July 2008, the details of which are shown in Figure 5.1 and Figure 5.2.

The panels were constructed from 100 mm thick glue laminated timber with 3 part finger joints at each corner screwed together with 200 mm long 8 mm diameter screws. The vertical steel was 10 mm diameter with the corner braces 12 mm diameter. Details of the panels are shown in Figure 5.1 and Figure 5.2.

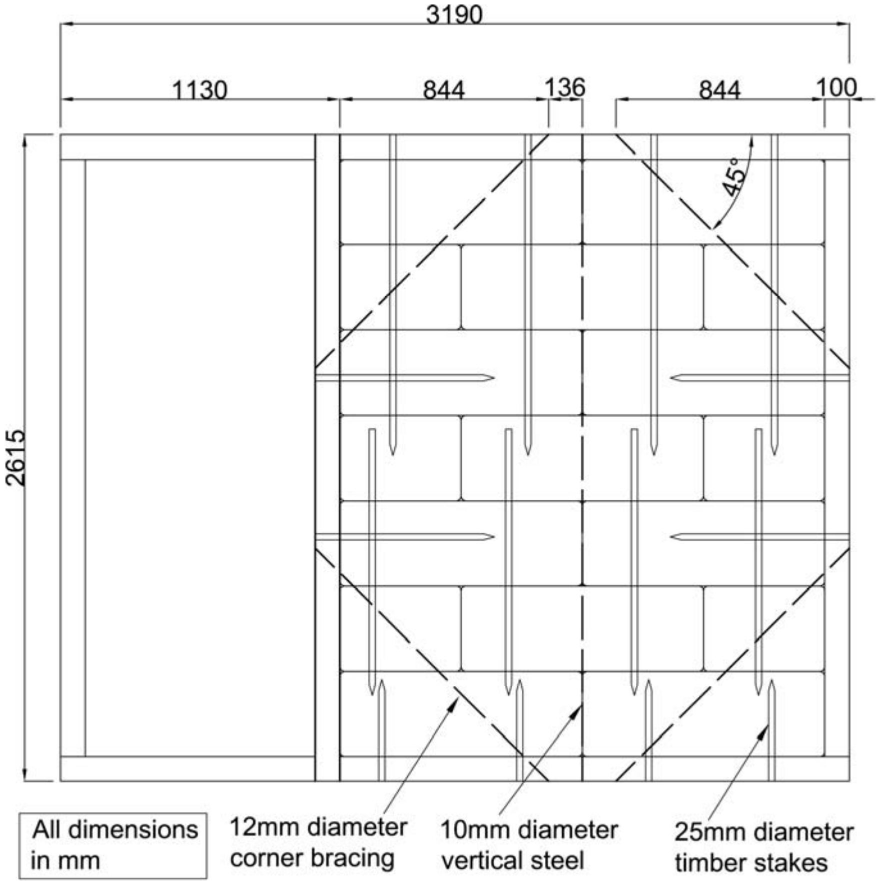

Figure 5.1 Grand Designs two bale panel

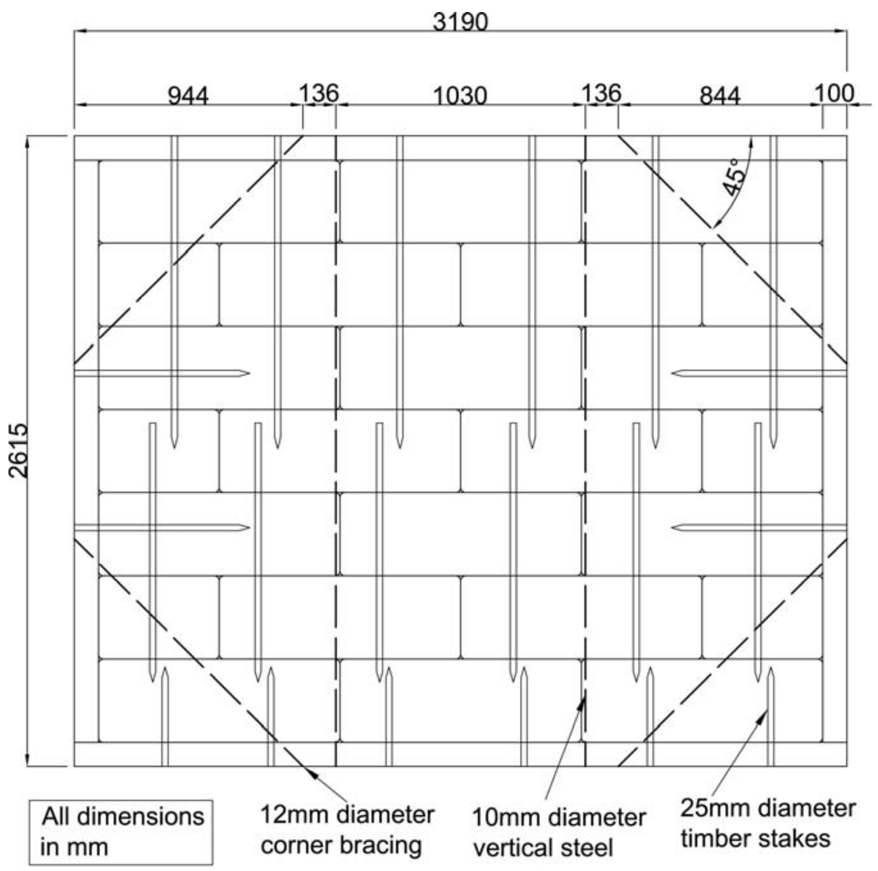

Figure 5.2 Grand Designs three bale panel

5.2.1 Initial Inspection

An inspection of the condition of the panels was undertaken upon arrival. Significant findings of this inspection are shown below:

Inspection carried out on 3/07/2008	
3 Bale Panel	
Inside face	Shrinkage cracks around render edges. Gap between render and timber (generally 1 mm or less). More severe along top edge and right hand side (up to 5 mm, see Figure 5.3). Some small denting in three locations. Plaster not cracked however, only surface damage. Some water staining at top of render (Figure 5.4). Four small cracks in render. Two at the bottom, one on the left hand edge and one on the top. They all seem to correspond to the location of the steel reinforcement and may be due to lifting.
Outside face	Small amount of shrinkage cracking in render around edges, less than 50 mm in length. Very small denting in places. Plaster not cracked, only surface damage. One long crack at top of panel near the reinforcement, 300 mm long. Possibly due to lifting (Figure 5.5).

2 Bale Panel	Vertical steel missing. Holes have been drilled in the timber but no bar fitted. Timber stakes missing in side of panel. It was not apparent if they were present on the end of the panel with the opening as the holes had been plugged. Upon deconstruction of the panel the stakes at the end with

	the opening were found to be present.
Inside face	Shrinkage of plaster away from timber frame, 1 to 2 mm on all edges. Crack at the top of the render near the reinforcement. May be linked to lifting of the panel. Water staining at top of panel. May be linked to the lack of steel reinforcement. As a result the hole in the timber is open at the top of the panel and water could enter here. 40 mm diameter indent/hole in bottom left corner. Has cracked the plaster and punched straight through the plaster but the straw is not visible (Figure 5.6).
Outside face	Large gap between plaster and timber at top of panel due to shrinkage. Smaller gaps elsewhere. Water staining at top, it may be due to missing vertical steel. Grass shoot starting to grow from crack between plaster and timber at top where worst water staining is (Figure 5.7). Two cracks at top of panel, one 400 mm long, the other 200 mm long (Figure 5.8.) Possibly due to lifting.

Figure 5.3 Gap between render and timber

Figure 5.4 Water staining

Figure 5.5 Crack in render

Figure 5.6 Hole in render

Figure 5.7 Grass growing from panel

Figure 5.8 Cracks in render

The moisture content of the bales in the panels was also surveyed when the panels were delivered. This was done by drilling two small holes in the top of the frame through which a Balemaster probe was inserted to different depths. The results are shown in Table 5.1 and Table 5.2. Note that 36.8% is the highest reading the Balemaster probe is able to record (readings above this value may therefore be indicative of much higher moisture levels).

Table 5.1 Arrival Moisture survey – 3 Bale Panel

Inspection carried out on 03/07/2008 **3 Bale Panel**		
Penetration depth (mm)	Moisture Content (%H_2O)	
	Hole 1	Hole 2
100	8.8	10.5
200	17.9	19.1
300	17.4	20.3
400	18.0	23.8
500	17.3	36.8
600	17.9	35.2

Table 5.2 Arrival Moisture survey – 2 Bale Panel

Inspection carried out on 03/07/2008		
2 Bale Panel		
Penetration depth (mm)	Moisture Content (%H_2O)	
	Hole 1	Hole 2
100	8.3	8.6
200	14.9	17.6
300	16.0	17.3
400	17.3	18.9
500	15.4	17.8
600	17.4	17.0

The moisture content in some locations is higher than the suggested 25% limit (Lawrence et al. 2009b) considered necessary to prevent decay of the straw. This occurred because the panels had been stored outside for a short period after removal from the Grand Designs Live site in London and delivery to Bath. It appears that water had entered the panels through gaps around where the timber stakes and bracing bars penetrate the timber frame. The high moisture content was unlikely to adversely affect the short term structural integrity or strength of the panels and therefore would not affect the racking shear testing being carried out on the panels. However in the long term it could be a significant problem as the straw would decompose at the interface with the render and this would reduce the adhesion of the render leading to buckling under load.

5.2.2 Racking shear testing

Both the Grand Designs ModCell panels were tested using the test set ups shown in Section 3.5. The three bale panel was loaded until the load cell recorded a load of 20 kN. The load was then removed and the residual deflection noted. This process was repeated at 30 and 40 kN. Following this the panel was unloaded whenever the render cracked. A peak load of 58.3 kN was recorded. A similar process was followed for the two bale panel at loads of 10, 15, 20 and 25 kN. A maximum load of 33.7 kN was recorded. Figure 5.9 shows the loading and unloading cycle as described above for both panels. It should be noted that throughout this thesis the loads applied during racking testing are displayed on graphs in kN/m which represents the load applied to the panel divided by its length. This allows for easy comparison between racking loads for panels of different sizes. The panels are non-linear elastic as with each cycle of loading there is some residual displacement. After being loaded to 20 kN (6.27 kN/m) the residual displacements for the two bale panel and three bale panel were 8.0 mm and 3.6 mm respectively. Once the render has cracked the residual displacement increases significantly.

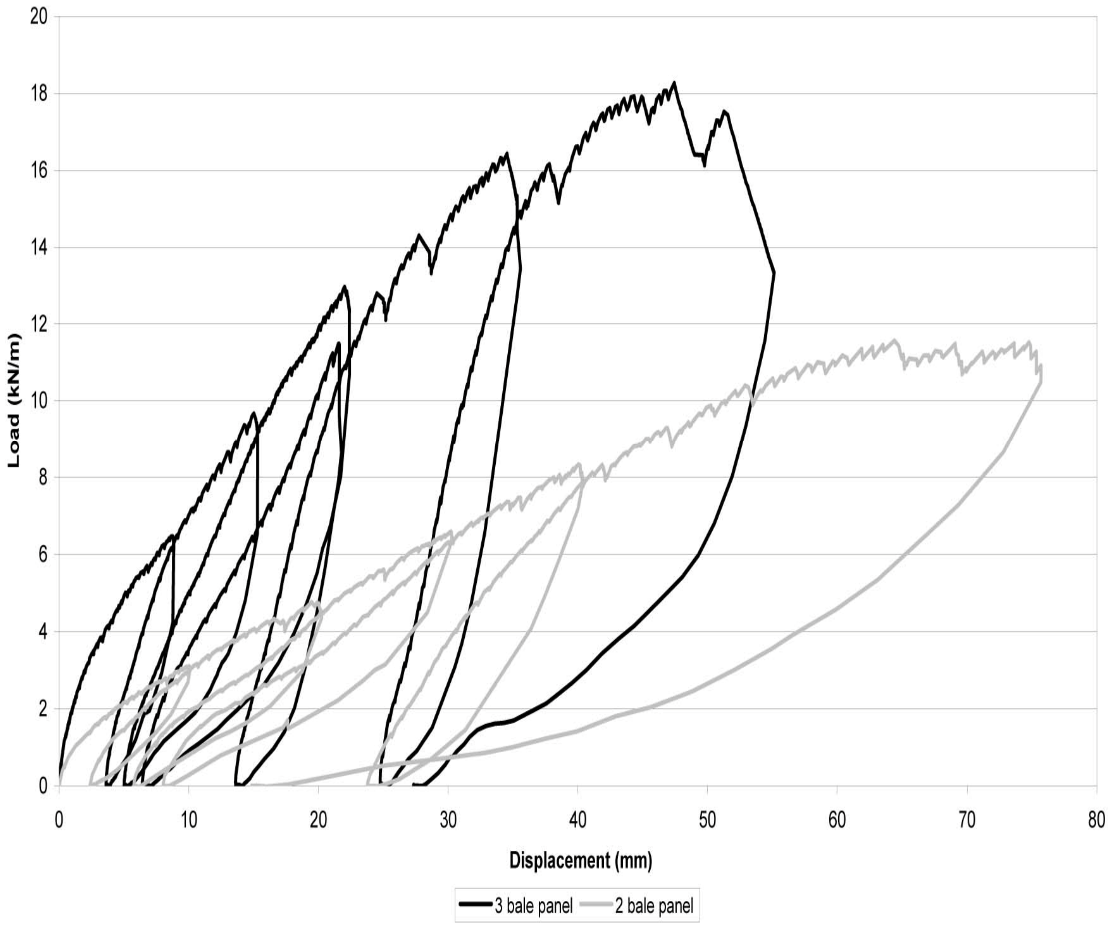

Figure 5.9 Loading cycle on Grand Designs panels

Figure 5.10 shows the results for both of the panels. The unloading cycles of the tests have been removed for clarity. It also indicates the horizontal deflection limits of h/500 and h/300 and when the first cracks in the render occurred for each panel. During the racking shear tests there is a tendency for the ModCell panels to rotate slightly about the bottom corner furthest away from the load application. Unfortunately on the three bale panel test there was no instrumentation to measure this global rotation, it was however measured on all subsequent tests. Therefore in Figure 5.10 the results for the three bale panel are not corrected for global rotation.

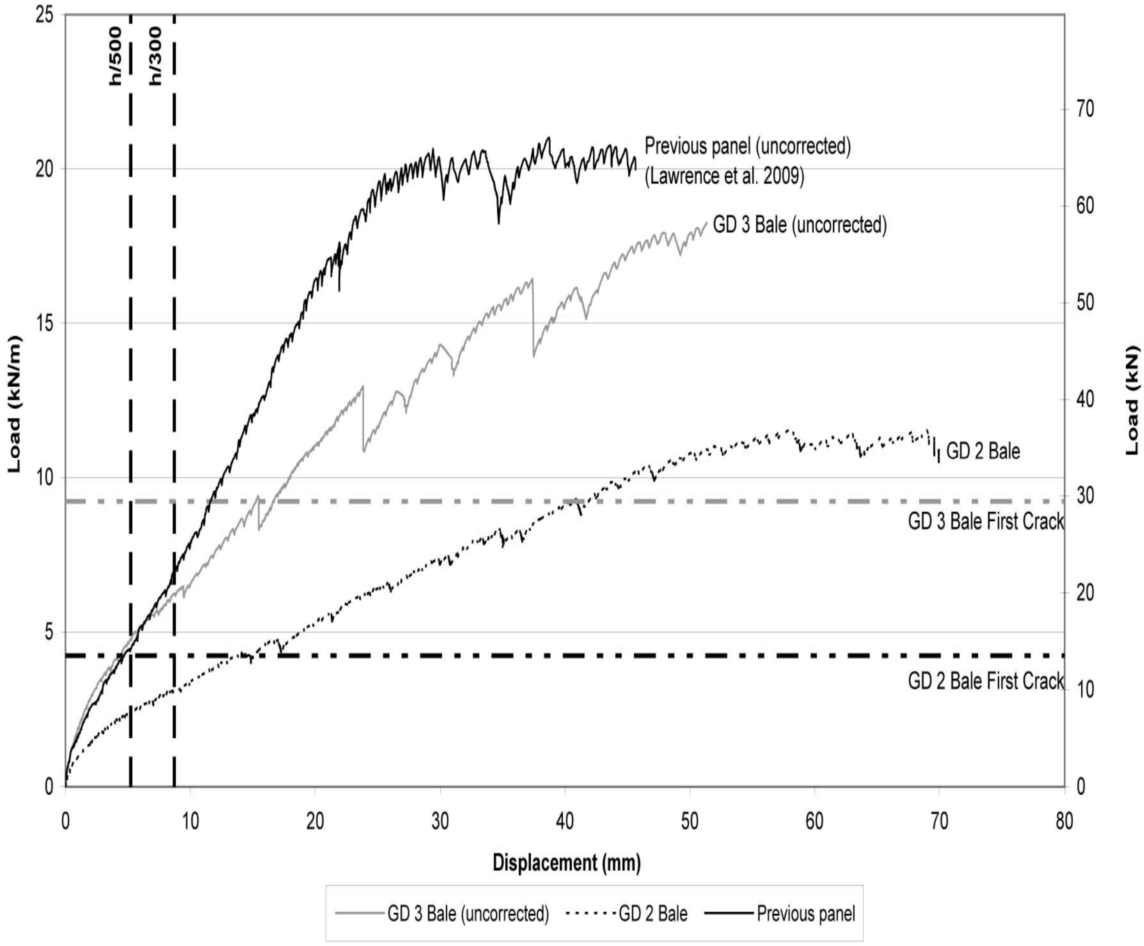

Figure 5.10 Grand Designs panels racking shear results

The results clearly show that the three bale panel is significantly stiffer than the two bale panel. At h/500 the three bale panel is carrying a load of 4.76 kN/m and the two bale panel is carrying 2.41 kN/m, which is roughly half the three bale panels load. There are several possible reasons why the two bale panel is less stiff. One reason for the lower stiffness of the two bale panel may be the arrangement of steel reinforcement. As shown in Figure 5.1 the two bale panel only has one vertical reinforcing bar in each face whereas the three bale panel has two. When the panels are loaded the load is applied to the end of the top element of the frame. As the load is transferred from the top element into the side of the frame nearest the jack there is a tendency for the side and the top of the frame to lift. The vertical reinforcement helps to resist this. Therefore with fewer reinforcing bars to resist this force the effects will be larger. Additionally the two bale panel only has two-thirds the area of render that the three bale panel. Whilst the panels are loaded in racking shear the top and the bottom of the frame move closer together and the sides rotate about their bases. This puts the render into shear eventually resulting in tension cracks. Therefore the panel with a smaller area of render will be able to carry a lower load.

The effects of fewer reinforcing bars and smaller render area were exaggerated on the two bale panel as it had been constructed onsite at Excel in London by celebrities and therefore the panel was built to a much lower standard of quality. As a result the vertical reinforcing bars had not been put in and some of the timber stakes were also missing. This will have affected the structural strength of the panel as the top frame element will have had significantly less restraint against uplift.

Figure 5.10 also shows the results for a previous ModCell panel racking shear test carried out by Lawrence et al. (2009a) at the University of Bath. This panel had different joint and bracing details and used 81mm thick cross laminated timber for the frame. From the graph it can be seen that the three bale Grand Designs panel was only very slightly stiffer than the previous panel at a displacement of h/500. After this point the previous test panel is stiffer. Initially this appears to be a surprising result as the Grand Designs panels were designed as load-bearing panels with stiffer frame elements, whereas the previous panel was designed as a cladding panel.

The following reasons are proposed to explain this outcome. Firstly, the bottom joints of the Grand Designs panels had fewer screws than specified. They were only pinned through the corners with one screw on each side instead of having eight screws going through one part of the joint into the end grain of the other part. Consequently during testing these joints failed prematurely. Additionally the panels from the Grand Designs house had been transported many more times and over much greater distances than normal for a ModCell panel. This had caused larger gaps to form between the render and the timber frame. Also lifting during transportation may have helped to weaken the panels as the timber frame deflects when being lifted resulting in load being put onto the render. Some cracks in the render surface prior to testing have been attributed to this. In contrast the panel tested by Lawrence et al (2009a) was not transported at all as it was constructed and tested in the laboratories at The University of Bath.

On both the two bale and three bale panels the first tensile cracks in the render appeared at a displacement of over h/300. Thus, within the serviceability limits (h/500) there was no visible damage to the panels. Following the racking shear tests samples of the render were taken from both the panels and tested to determine the compressive and flexural strength following the method in Section 3.3 of this report. For the two bale panel the average compressive and flexural strengths were 2.12 N/mm^2 and 1.14 N/mm^2 respectively. For the three bale panel the average compressive and flexural strengths were 2.43 N/mm^2 and 1.31 N/mm^2 respectively. The full results from these tests are included in Table 5.3. The two bale panel was rendered on site during the construction of the Grand Designs house rather than in the flying factory. This is likely to be the reason why the render strengths were lower. Figure 5.11 and Figure 5.12 show the panels after racking shear testing had been completed. The cracks in the render surface have been marked so they can be easily identified.

Table 5.3 Lime render specimen test results

Specimen	Flexural Strength (N/mm^2)	Compressive Strength (N/mm^2)
3 Bale No. 1	1.10	1.99
		2.07
3 Bale No. 2	1.42	2.29
		2.16
3 Bale No. 3	1.75	2.47
		2.22
2 Bale No. 1	1.19	1.80
		2.00
2 Bale No. 2	0.95	1.95
		1.72
2 Bale No. 3	1.26	2.17
		2.14

Figure 5.11 Two bale Grand Designs panel following testing

Figure 5.12 Three bale Grand Designs panel following testing

5.2.3 Post Test Inspection

Following testing both panels were intrusively inspected in order to fully establish the moisture content of the bales and the thickness of the render (Figure 5.13). The results of the moisture content survey are shown in Figure 5.14 and Figure 5.15. The numbers shown are an average of the moisture content from each of the faces of the panel just behind the render and the moisture content in the centre. Moisture was taken on 4th August 2008 using a Balemaster probe and is recorded in %H_2O. Note that 36.8% is the highest reading the Balemaster probe is able to record (readings above this value may therefore be indicative of much higher moisture levels). This survey of the bale moisture content shows similar results to the one carried out when the panels arrived. The areas near to the holes in the frame for the timber stakes and braces again showed the highest moisture levels confirming that these areas were the most likely point of entry for the water.

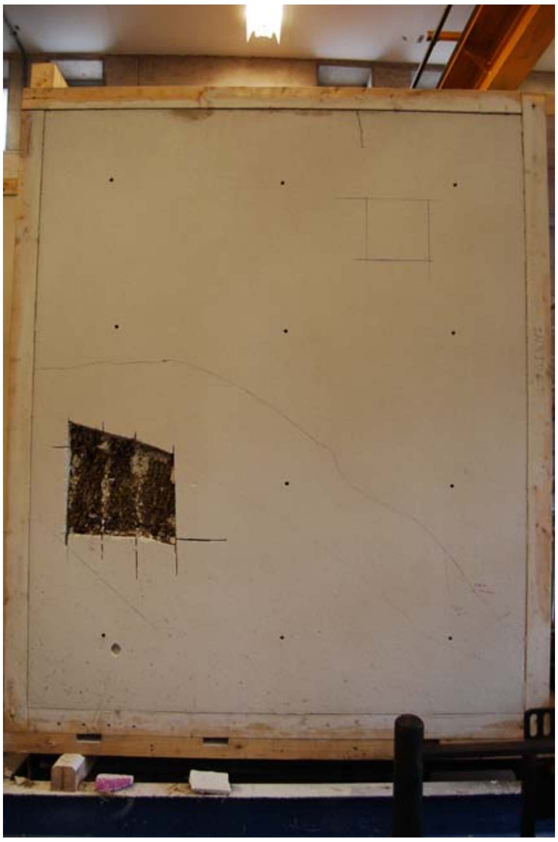

Figure 5.13 Two bale Grand Designs panel during moisture and render survey

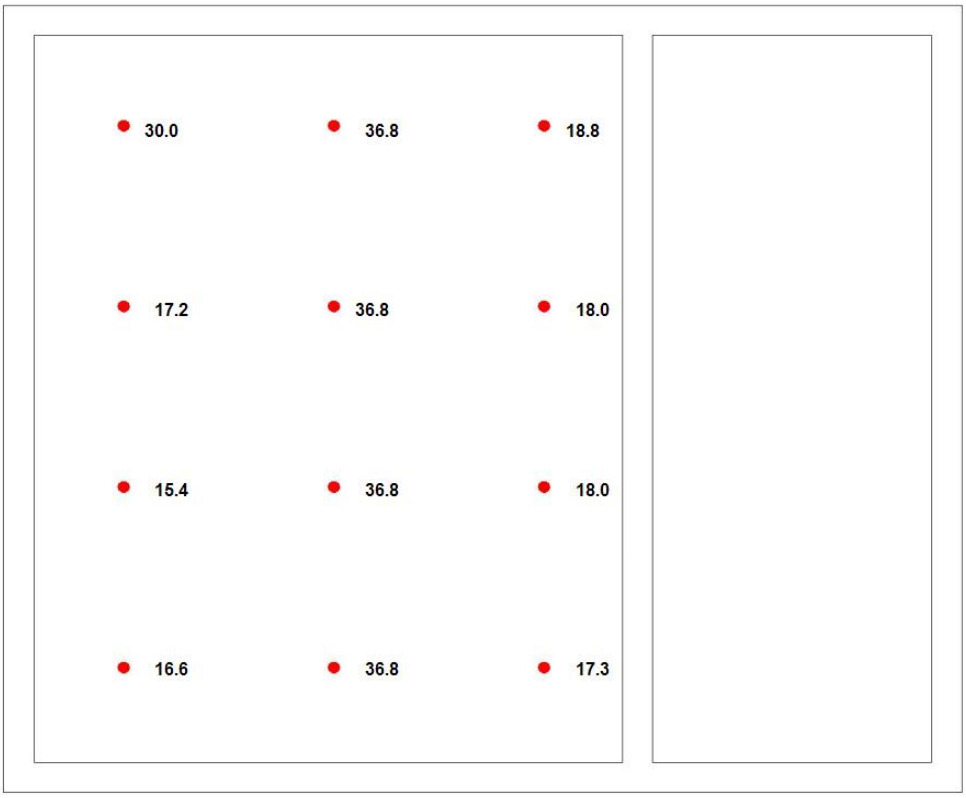

Figure 5.14 Two bale Grand Designs panel moisture readings (%H_2O)

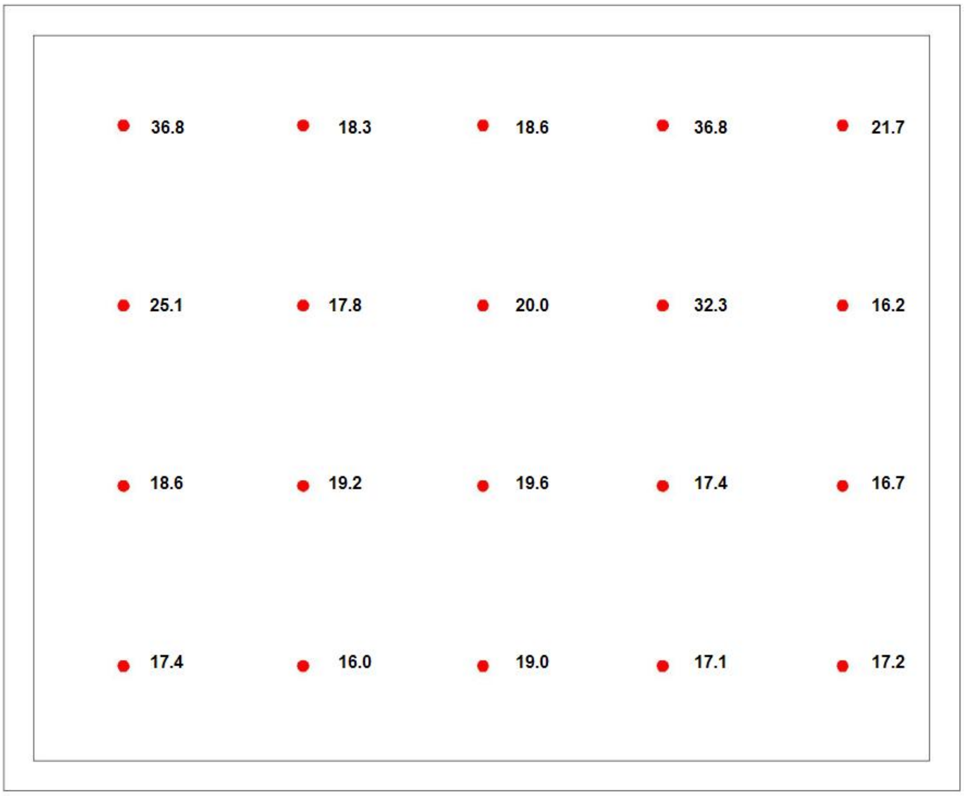

Figure 5.15 Three bale Grand Designs panel moisture readings (%H_2O)

The survey of render thickness revealed two notable things. Firstly it showed how the thickness of the render varies across the face of the panels. The render varied in thickness from 15 mm to over 40 mm. The render is specified to be 30 mm thick as this has been found to provide adequate protection to the straw. Where the render is thinner than this degradation of the straw may become an issue in the future. Secondly it showed how well the render bonds to the straw. The render bonds very well as it is applied to cut straw ends and it is therefore able to penetrate into these ends well. It is also spray applied which increases the penetration further by forcing the render into the bale face.

Once the moisture and render inspections had been completed the panels were dismantled in a controlled manner in order to inspect the internal condition of the straw. Upon deconstruction the areas that water had penetrated could clearly be seen. There was evidence of water ingress around joints and openings in the top timber plate, for example where the stakes had been driven through and the reinforcement holes (which were open on the 2 bale panel). Water ingress was also seen around the render edges and at the corner joints (Figure 5.16).

Figure 5.16 Top corner joint from two bale Grand Designs panel showing signs of water ingress

The water ingress and deterioration of the straw continued down through the panel, especially at the location of the reinforcement (Figure 5.17). Only the top one or two bales were affected from the holes where the stakes had been driven as not as much water had penetrated at these locations. Some of the stakes where water ingress had been most severe had signs of mould growth on them.

Figure 5.17 Two bale Grand Designs panel showing signs of water ingress along line of reinforcement

Upon deconstruction of the corner joints it became apparent that some of the screws in the bottom joints were shorter than 200mm. This will have affected the overall stiffness of the panels and is another reason why the bottom joints failed prematurely in the racking shear tests.

5.2.4 Conclusions from Grand Designs Panels

In conclusion the Grand Designs panels were not a true representation of load-bearing ModCell panels due to poor construction and excessive transportation. The tests did however show that if the joints are not adequate they will pull apart vertically causing a premature failure of the panel. If bottom joints had been constructed as the top joints (eight 200 mm long screws) then the panels would have been stiffer and stronger. During testing the top joints on both panels did not show any signs of damage or failure.

These tests have shown that the racking shear resistance of two bale panels is less than that of three bale panels. It is the first time that a two bale panel has been tested and therefore further testing of two bale panels will be necessary in order to determine if this is an anomaly due to the poor construction and missing reinforcement. The effect of a window or door within the opening in the two bale panel should be considered, however its strength should not be relied upon.

The moisture surveys and deconstruction of the panels have highlighted the problems that water ingress can cause. Water ingress during construction, transportation or storage of the panels could cause major problems throughout the life of the building. The evidence found during the post test inspections shows the short term damage a small amount of water can cause. If the straw close to the render surface decays then the render will no long be securely attached to the straw. This could adversely affect the strength and stiffness of the panels as the render makes a large contribution to this and is restrained from buckling by the straw. Therefore making sure the panels are properly covered at all times before the building is water tight is important.

Overall these panels have provided some useful results and highlighted some problems, but they did not show the full potential of load-bearing ModCell panels.

5.3 Laboratory Prepared Panel Tests

The Grand Designs panels were not a true representation of load-bearing ModCell panels and therefore another series of full size racking shear tests were carried out. Also the Grand Designs panels were constructed prior to the joint testing and therefore the conclusions from this were not implemented in their design. Further testing will allow for this and also allow for the true performance of load-bearing panels to be shown. The panels were constructed by the ModCell team in the structures laboratory at the University of Bath where they remained until they were tested. The Limetec Basecoat render was spray applied in two layers with 48 hours between each application. Once the full build up of render had been applied the panels were left for 14 days before testing. The panels were tested using the set ups shown in Figure 3.5 and Figure 3.6 for the two bale and three bale panels respectively.

5.3.1 Panel design

The design of the panels was informed by the results of the joint and panel tests discussed previously. Four panels were tested, two, three bale panels and two, two bale panels. Corner bracing was used in one two bale panel and one three bale panel, the two panels had cross braces. The corner braces were increased in angle from 45^o to 51^o. The use of cross bracing has several disadvantages. Cross bracing requires reinforcing bars over 3 meters in length which are not readily available and are therefore more costly. It also requires thicker render as the bracing and vertical bars cross. Therefore the render has to be thicker in order to maintain the minimum render cover over the reinforcing bars. However if the required structural stiffness for load-bearing construction could only be achieved through using a cross brace, it is preferable to do this than not to be able to provide load-bearing panels at all. Therefore it was felt that it was important to investigate if cross bracing the panels gave any significant structural advantage. Theoretically cross bracing a frame of any kind is the most efficient way of preventing horizontal displacements. All of the panels also had vertical reinforcement apart from the two bale cross brace panel. If vertical reinforcement had been used in this panel then three reinforcing bars would have crossed at the same point. In order to provide the minimum render cover to the reinforcement the render would have had to be 40% thicker than on the three bale cross brace panel. This is an inefficient use of material and would also add significant amount to the cost of the panel. Details of all the panels are shown in Figure 5.18 to Figure 5.21.

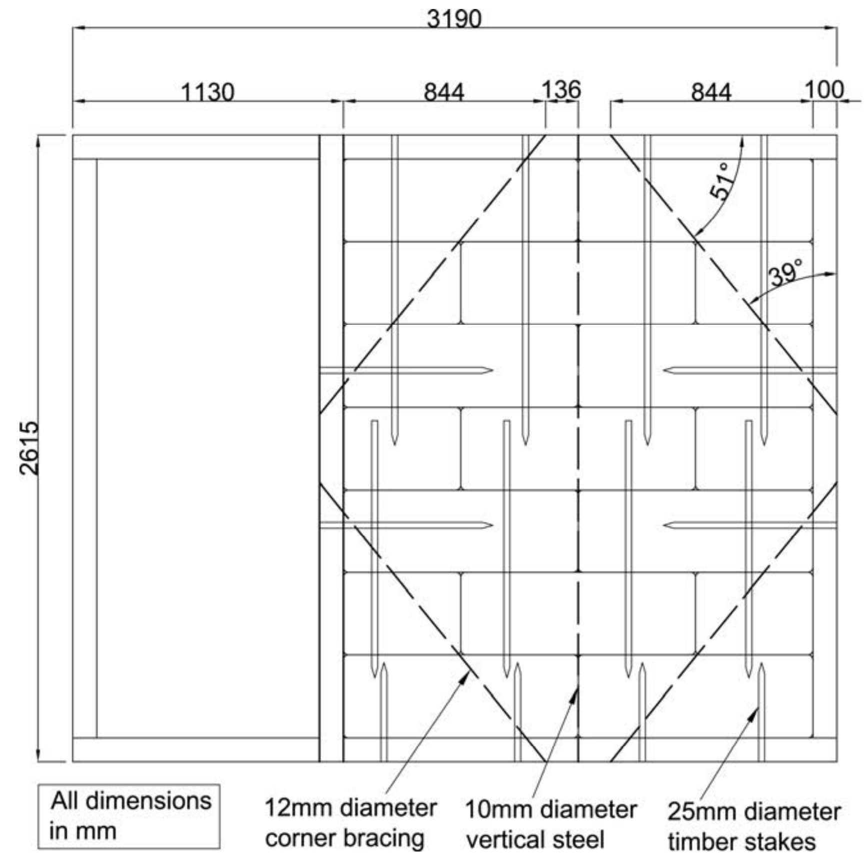

Figure 5.18 Two bale corner brace panel

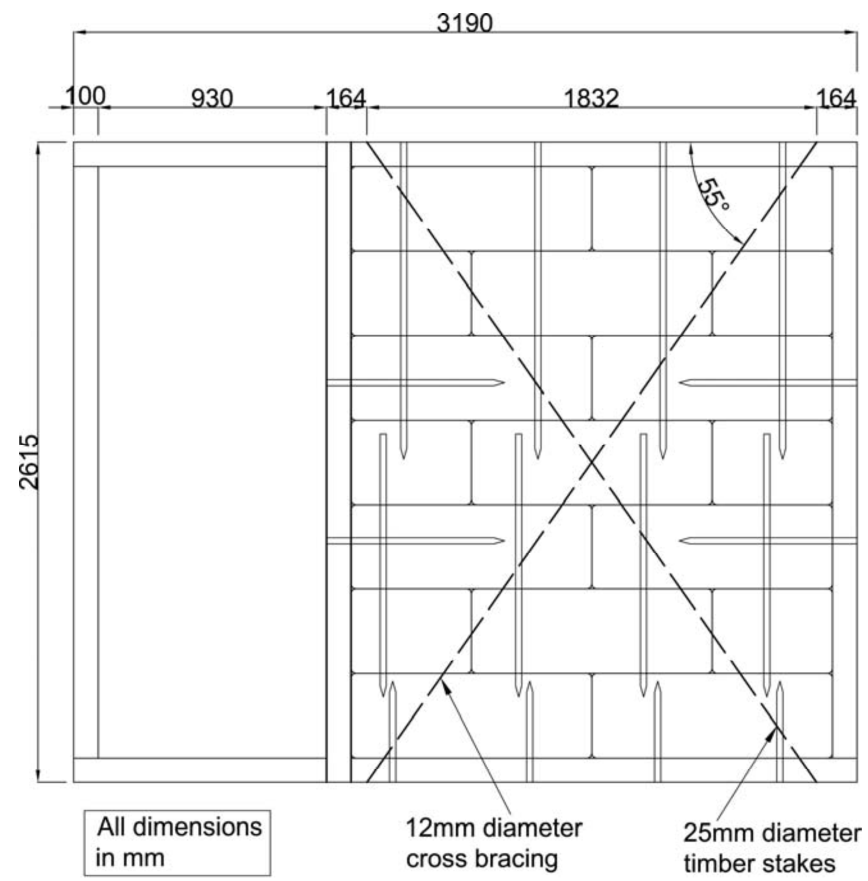

Figure 5.19 Two bale cross brace panel

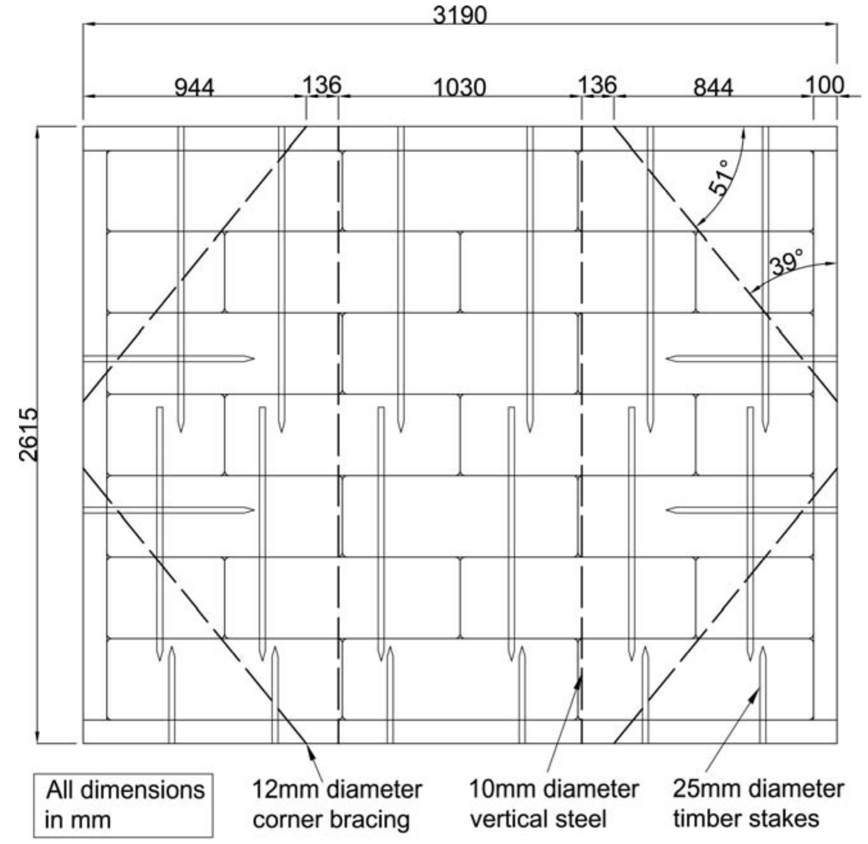

Figure 5.20 Three bale corner brace panel

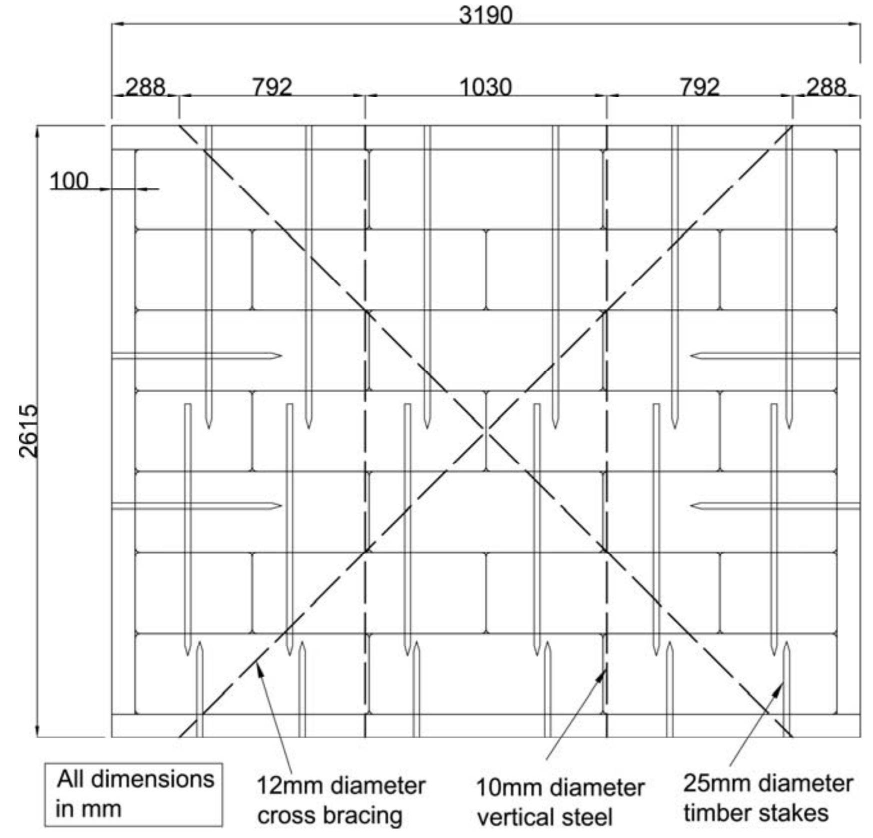

Figure 5.21 Three bale cross brace panel

5.3.2 Racking shear testing

All of the panels were initially loaded until a horizontal displacement of h/500 was recorded, then the load was released and the residual displacement recorded. The process was repeated for displacements of h/300 and h/100. Following this the panels were loaded until cracking occurred in the render at which point they were unloaded and the reloaded until failure. Figure 5.22 shows the loading and unloading cycle for two of the panels tested. The other two panels showed similar results. As with the Grand Designs panels, these panels are not linear elastic as with each cycle of loading there is some residual displacement. The residual displacement after loading to an indicated h/500 is 1.90 mm for the two bale corner brace, 2.20 mm for the two bale cross brace, 2.19 mm for the three bale corner brace and 2.49 mm for the three bale cross brace. These displacements are acceptable as they are minimal and not noticeable. Once the render has cracked the residual displacement increases significantly.

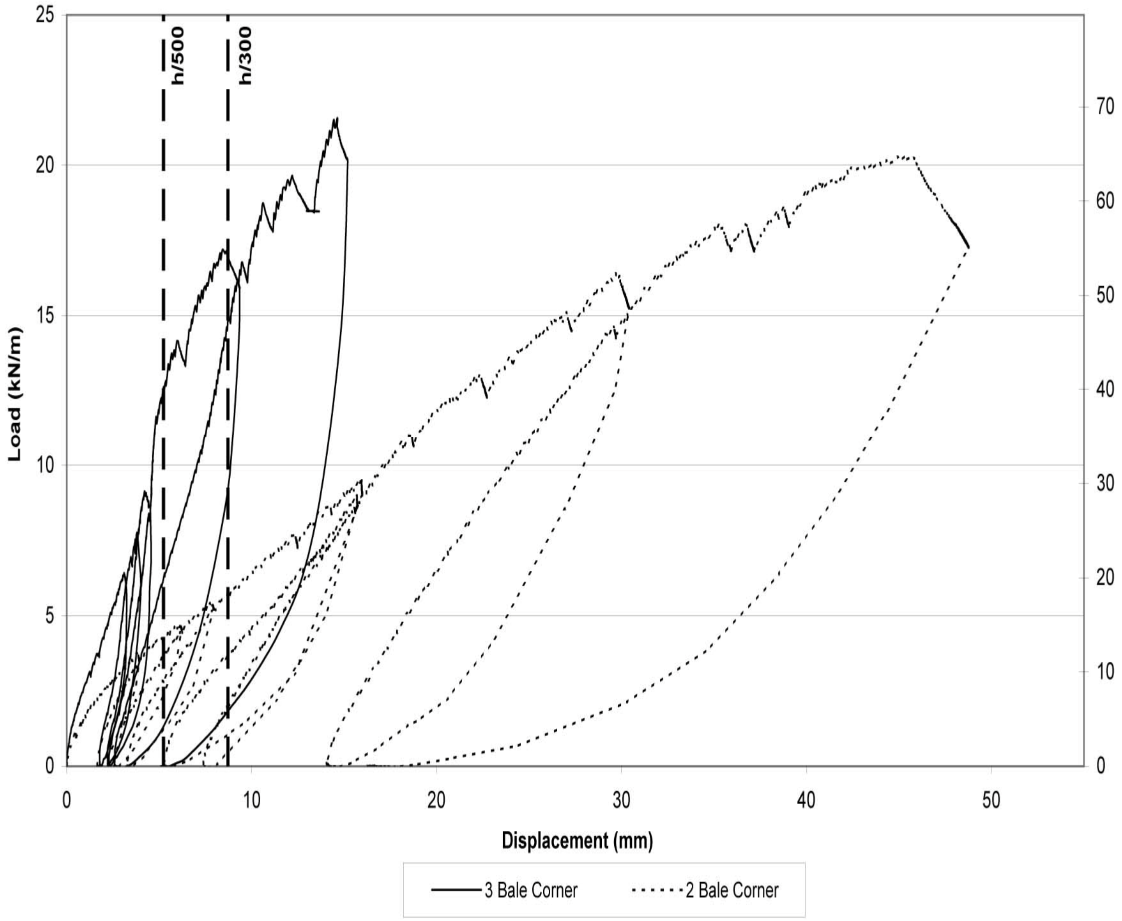

Figure 5.22 Typical loading and unloading cycles

During testing all of the panels followed the same general trend. Initially as they were loaded the shrinkage gaps between the render and the timber frame either opened or closed depending on where they were on the panel (Figure 5.23). As the load was increased further the timber fame started to load the render surface in shear. The render showed some signs of local crushing where the frame was bearing on it. With increasing load cracks begin to form in the render. The render typically cracks in a diagonal pattern across the centre indicating a tensile failure (Figure 5.24). As the load continues to be increased further cracks form and the gaps between the render and frame continue to open up further.

Figure 5.23 Gaps opening between timber and render

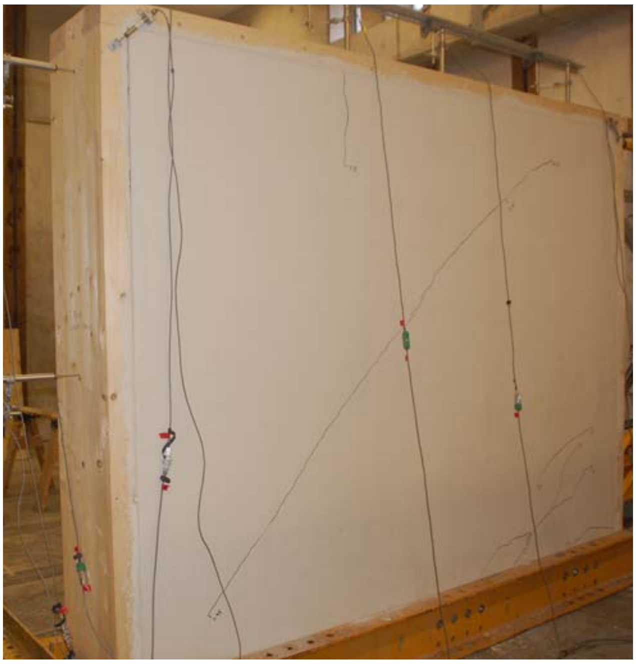

Figure 5.24 Diagonal crack in render

It is difficult to define panel failure when testing. As the main consideration when designing ModCell panels is their performance under serviceability loading, the performance at a displacement of h/500 and the load at which the render cracks are important. Therefore for the two bale cross braced and three bale cross braced panels testing was stopped when the hydraulic jack reached the end of its stroke. For the two bale corner braced and three bale corner braced panels testing was stopped at a recorded horizontal displacement of 39 mm as the panels were not significantly damaged so will be reused to further the development of load-bearing ModCell panels. Figure 5.25 shows the results of the racking tests.

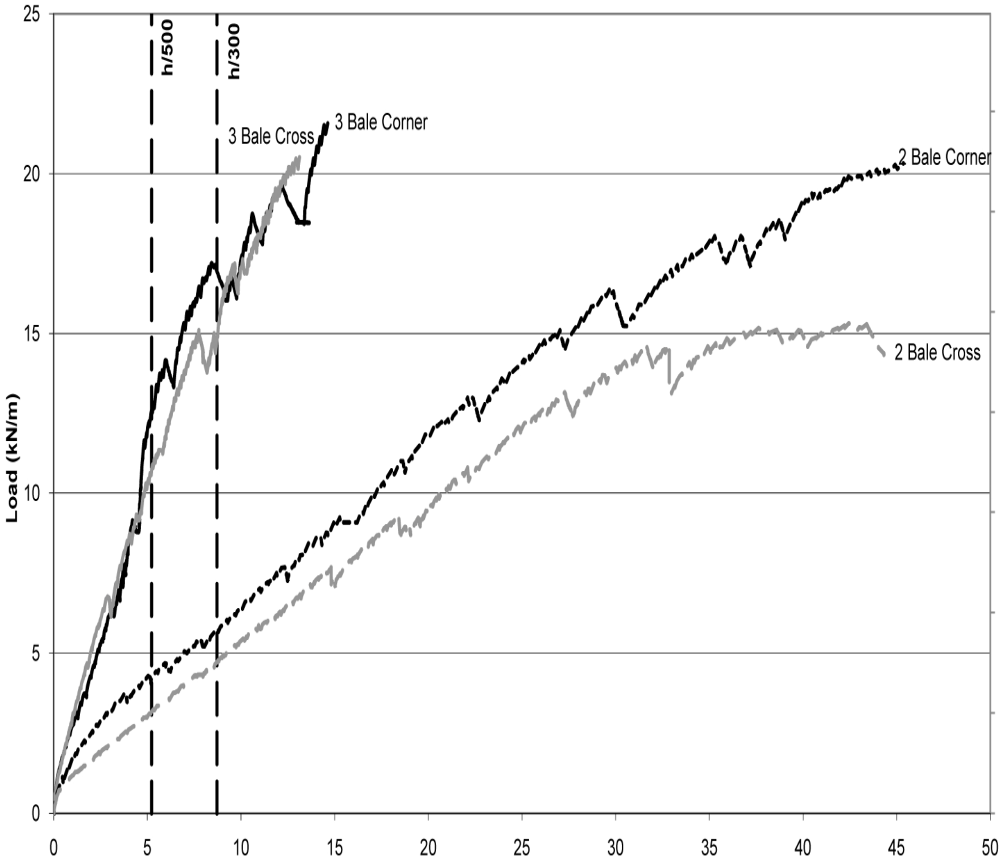

Figure 5.25 Further racking shear test results

Initially the three bale panels are over three times stiffer than the two bale panels, as shown in Figure 5.25. At a displacement of h/500 the load carried by the three bale corner braced panel is 12.62 kN/m; by the three bale cross braced panel is 10.71 kN/m; by the two bale corner braced panel is 4.16 kN/m; by the two bale cross braced panel is 3.14 kN/m. The reasons why the three bale panels are stiffer than the two bale panels are the same as those proposed in Section 5.2.2 of this thesis. In summary these were; that the two bale panels have fewer vertical reinforcing bars to resist vertical displacement of the top frame element; and that the two bale panels have a smaller area of render.

During testing of the panels all of the joints remained intact. This suggests that the new screw connected joint is adequate in terms of vertical pull out. Partly as a result of the joints remaining intact there was considerable global rotation of the panels during testing about the bottom corner opposite the load point. The global rotation has been corrected for in the results shown in Figure 5.25.

In both the two and three bale panels the corner braced panels out performed the cross braced panels in terms of stiffness and strength. This is initially surprising and a potentially unexpected result. The following reasons are proposed to explain this. With the two bale panels this may have been related to the vertical reinforcement. In the corner braced panel there was one vertical reinforcing bar in each face, whereas in the cross braced panel there were none. As previously mentioned in connection with the Grand Designs panels the vertical reinforcement helps to restrict vertical movement of the top element of the timber frame during loading. The top frame element in the cross braced panel would have had less resistance to uplift without the vertical reinforcement, hence the corner braced panel is slightly stiffer as the reinforcement was present.

In both the two and three bale panels the increased stiffness of the corner braced panels may be due to the way in which the panel works compositely when loaded. Plotting the deformed shapes shows that when the cross braced panel is loaded one of the bracing bars goes into tension and the frame changes from being a rectangle into a parallelogram (Figure 5.26). In this system the stiffness of the timber elements does not contribute to the overall stiffness of the frame. In the corner braced frame, at the diagonally opposing corners that are trying to open the bracing bars develop tension. The bracing prevents the corner opening and therefore in order for the frame to deflect the timber starts to bend (Figure 5.26). This will add considerable stiffness to the frame as the cross section of the timber is large. Therefore while a cross bracing system is generally more efficient than corner bracing, in this particular case when the corner bracing is combined with the stiffness of the timber it out performs the cross bracing system.

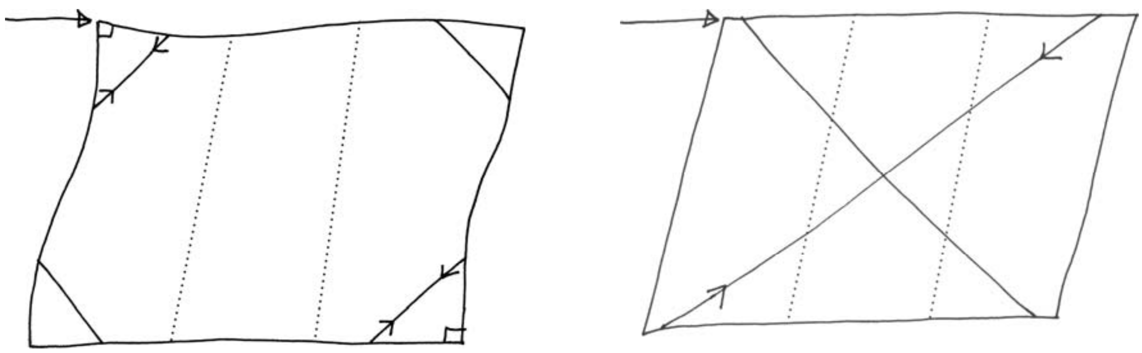

Figure 5.26 Deformed panel shapes

It has already been commented that none of the joints failed during testing. Therefore due to the increases in joint strength it should be considered what other parts of the panel are likely to fail or have failed. Following testing there was no visible damage to the panels other than cracking to the render. Upon closer inspection there was some cupping of the washers to the ends of the bracing bars. This indicated that the reinforcing bars were under considerable load during testing, however they did not fail.

During the racking shear testing the render cracked at a horizontal displacement of h/280 (9.3mm) for the three bale corner braced panel; h/275 (9.5mm) for the three bale cross braced panel; h/220 (11.9mm) for the two bale corner braced panel; and h/100 (26.2mm) for the two bale cross braced panel. The factor of safety for when the first crack occurred for each panel has been calculated by dividing the crack load by the load at a deflection of h/500. The factor of safety for the three bale corner braced panel is 1.30; for the three bale cross braced panel is 1.58; for the two bale corner braced panel is 1.78; for the two bale cross braced panel is 4.11. This confirms that the render should not crack at the design serviceability load deflections as the cracking loads are at least 30% higher than the loads at a displacement of h/500. The cracking of the render in all of the panels followed the patterns that were expected with diagonal tension cracking across the face of the render as it was sheared. Figure 5.27 to Figure 5.30 show the cracking in the render of the panels at the end of each test. Any cracking around the base of the panels is associated with the how the panels were fixed to the laboratory floor and is not directly caused by the load applied to the panel.

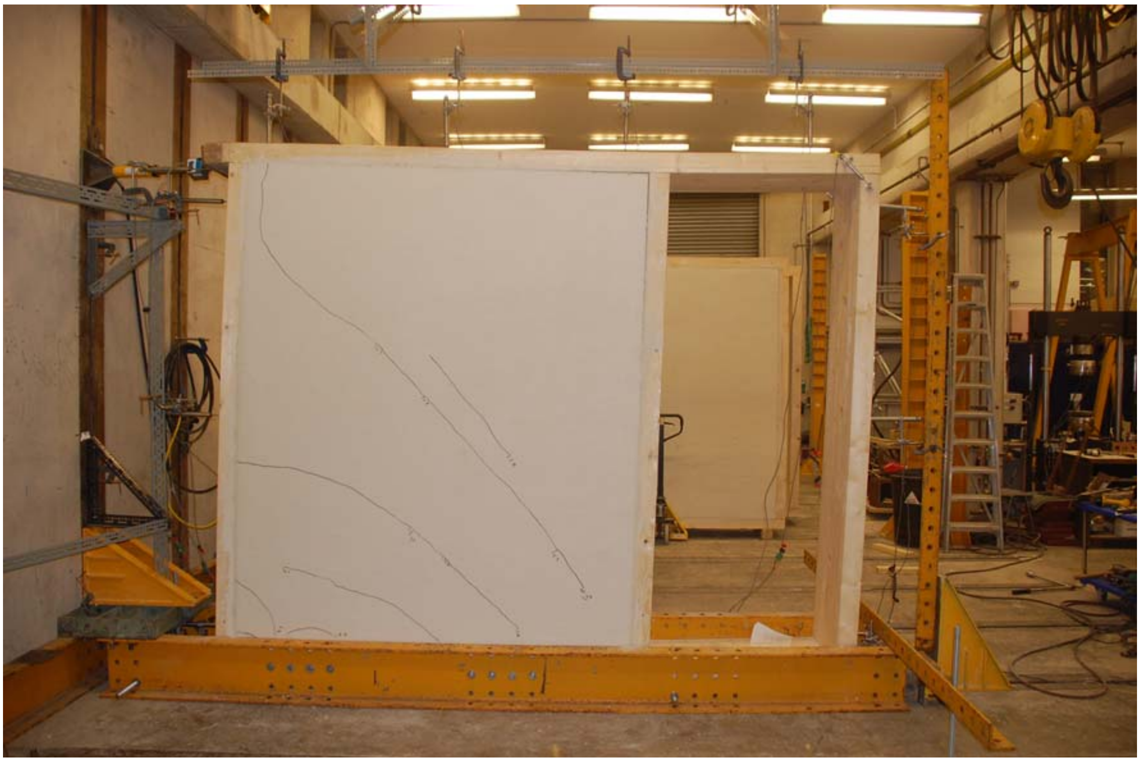

Figure 5.27 Two bale corner braced panel after testing

Figure 5.28 Two bale cross braced panel after testing

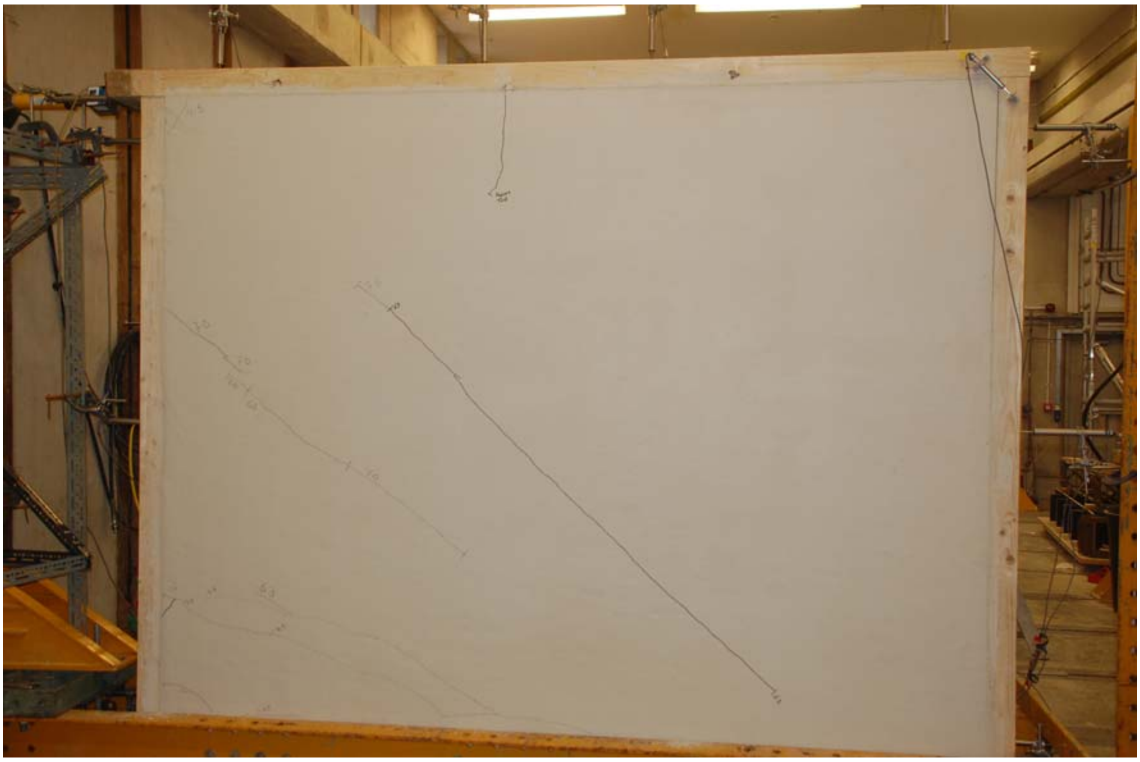

Figure 5.29 Three bale corner braced panel after testing

Figure 5.30 Three bale cross braced panel after testing

Table 5.4 shows a summary of the performance of all of the panels tested during this investigation and the performance of one previous panel.

Table 5.4 Results summary for all panels tested

Panel	Load at h/500 (kN/m)	Stiffness at h/500 (kN/mm)	Render cracking	
			Load (kN/m)	Displacement
Three Bale Corner	12.62	7.70	16.50	h/280
Three Bale Cross	10.71	6.53	16.90	h/275
Grand Designs Three Bale*	4.76	2.90	9.22	h/150
Two Bale Corner	4.16	2.54	7.39	h/220
Two Bale Cross	3.14	1.92	12.92	h/100
Grand Designs Two Bale	2.41	1.47	4.24	h/175
Previous Panel* (Lawrence et al. 2009a)	5.24	3.20	N/A	N/A

*Panels not corrected for global rotation

From the results shown in Table 5.4 it can be seen that the three bale corner braced panel is the stiffest and strongest under at the serviceability deflection limit of h/500. It is also over twice as stiff as the previous panel. The improvement in racking shear stiffness is as a result of several elements that have been revised on the ModCell panels since the work by Lawrence et al. (2009a). Firstly the timber frame has been increased in thickness to 100 mm in order to increase its stiffness and vertical load carrying capacity. Secondly the frame joints have been revised following the results gained from testing carried out during this investigation.

During construction of the panels samples of the render were taken. These were then tested at the same time as the panel to find the compressive and flexural strengths following the method outlined in Section 3.3 of this report. The four panels were tested over 3 days, as were the render specimens. The average flexural and compressive strengths of the render were 1.21 N/mm^2 and 2.85 N/mm^2 respectively. The details of the specimens tested and the results from these tests are shown Table 5.5.

Table 5.5 Render specimen test results

Specimen	Age (days)	Flexural Strength (N/mm^2)	Compressive Strength (N/mm^2)
No. 1	14	1.22	2.77
			2.94
No. 2	14	1.28	2.70
			2.86
No. 3	14	1.13	3.02
			2.85

5.3.3 Conclusions

From these results the three bale corner braced panel clearly performs best out of the three bale panels and the two bale corner braced panel performs best out of the two bale panels. This shows that corner bracing is adequate and that cross bracing the panels offers little structural advantage. This series of racking shear tests have confirmed that the results from the tests on the Grand Designs panels that two bale panels are less stiff than three bale panels. However further investigation into the effect doors and windows would have is necessary in order to fully understand the differences in performance between the two types of panel.

It has been shown that the residual horizontal deflections of the panels after they have been loaded to h/500 are within acceptable limits. However the effects of repeated loading to h/500 have not been investigated. This may be an issue that requires further investigation.

This series of racking shear tests has shown that a great improvement in racking shear strength has been achieved. The three bale corner braced panel is more than twice as stiff as the previous three bale panel tested by Lawrence et al. (2009a). It was expected that it would be stiffer as unlike the previous panel the three bale corner braced panel was specifically designed for load-bearing applications, but being over twice as stiff is a very welcome result.

The total racking shear load caused by the effects of wind on a typical two storey Balehaus made with load-bearing ModCell panels with a plan area of 6.8 metres by 6.8 metres is in the region of 35 kN. This is equivalent to a load of 2.6 kN/m along the two sides of the house parallel to the wind direction. The results from these racking shear tests have shown that a combination of the two and three bale corner braced panels have the required stiffness to withstand this racking shear force. For a three storey house with the same plan area the total racking shear load will be in the region of 3.9 kN/m along the two sides parallel to the wind direction (52 kN total). Therefore, with careful selection of where two or three bale panels are used within each elevation a three storey house is achievable in load-bearing ModCell panels.

5.4 Vertical Compression Testing

In a load-bearing ModCell building the floor is fixed directly to the top of the panels and therefore all floor loads have to be carried across the top of the panels to the sides of the frame. While it is simple enough to calculate the cross section of timber required to carry the floor loads, the effect any deflection of the timber will have on the straw and render below is less simple to work out. Therefore a vertical compression test on the straw and render was carried out to determine the effects loading the straw and render.

5.4.1 Testing

Only one test was completed as only one panel was available to be used for testing. Details of the ModCell panel are shown in Figure 5.31. The panel was originally designed and used to investigate the effects of continual exposure to the weather over the course of a year in a related research project. The panel was constructed from 81 mm thick cross laminated timber; it was one bale wide by five bales high. It has timber stakes pinning the bales both vertically and horizontally. The horizontal stakes were removed prior to testing so that they did not interfere with the compressive strength of the straw and render. The top joints of the panel were cut and the sides were restrained to prevent bulging. Figure 5.32 shows the results from this test. The graph plots load against average displacement of the top of the panel.

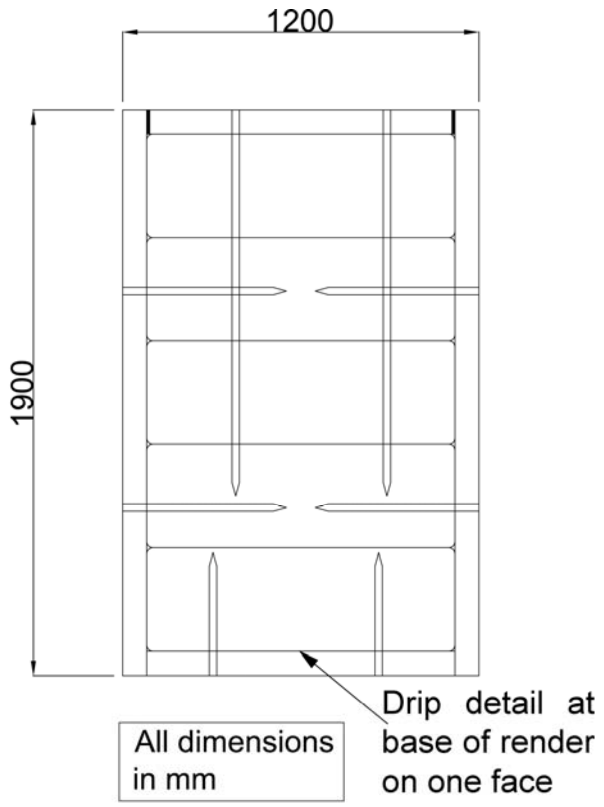

Figure 5.31 ModCell panel used in vertical compression testing

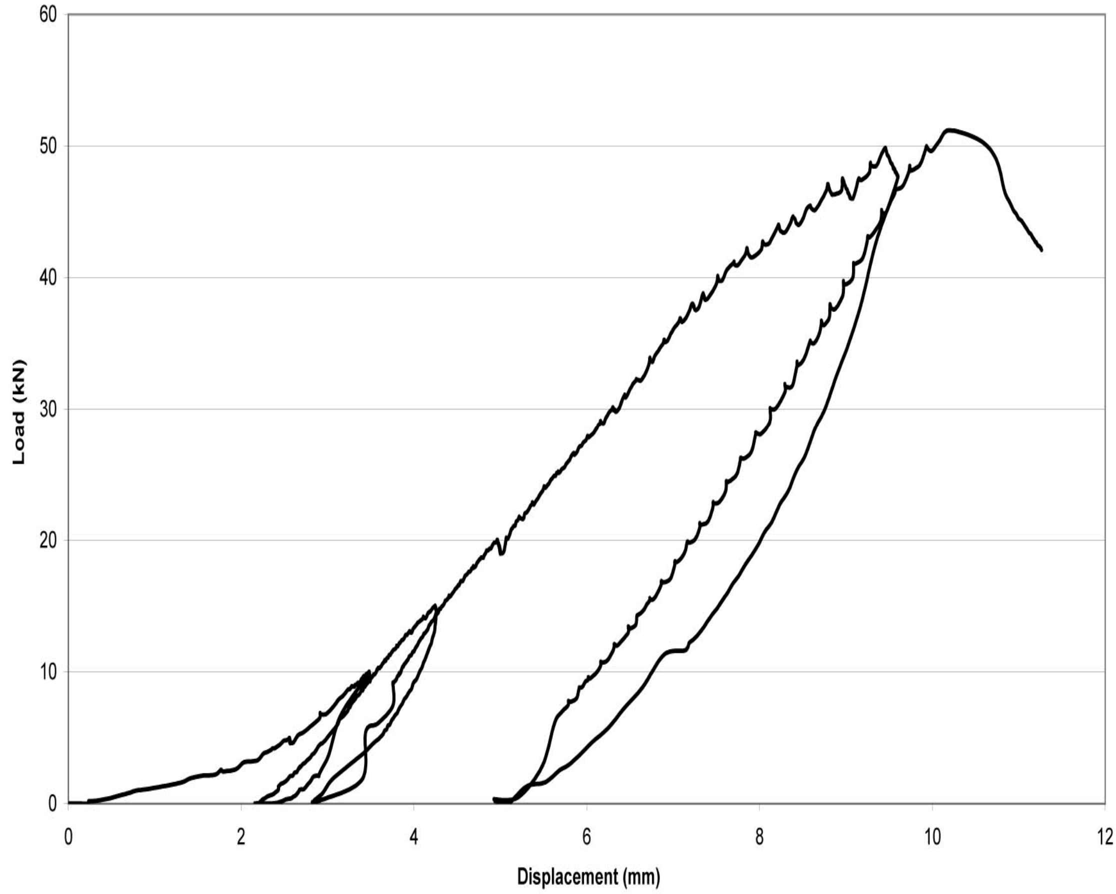

Figure 5.32 Results from Vertical compression testing

As the panel was loaded there was an initial settlement of the straw and render by 2 mm due to the shrinkage gaps between the timber and render at both the top and base of the panel. The panel was loaded to 10 kN, then unloaded and a residual displacement of 2.16 mm was recorded. The panel was then reloaded to 15 kN, unloaded and a residual displacement of 2.83 mm was recorded. The panel was then loaded to 50 kN, unloaded and a residual displacement of 4.95 mm was recorded. The panel was then loaded to failure. Following the initial displacement of 2 mm the stiffness of the straw and render remained constant throughout the test at 8.6 kN/mm.

As the load was applied the render slid over the drip detail on the external face of the panel (Figure 5.33). On the internal face the render is fully restrained by the timber frame and therefore the render was not able to slide past this. On this face the render deformed outwards into a curve down the panel's length (Figure 5.34). The panel failed by the render on this face cracking at mid height at a load of 51.2 kN and a deflection of 10.2 mm (Figure 5.35).

Figure 5.33 Render sliding over drip detail

Figure 5.34 Render curving under load

Figure 5.35 Failed render surface after testing

Following testing a survey of render thickness and moisture content was undertaken. The results are shown in Figure 5.36, Figure 5.37, Table 5.6 and Table 5.7. The render was an average thickness of 30 mm.

Panel side 1

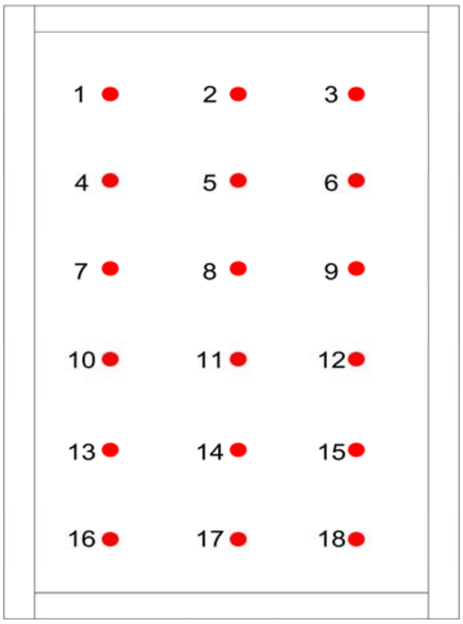

Figure 5.36 Render investigation locations side one

Table 5.6 Vertical compression panel render thickness and moisture content – side one

Location	Render Thickness (mm)	Moisture content % H_2O		
		Front	Middle	Back
1	20	12.8	13.8	14.6
2	33	14.1	14.0	13.9
3	31	13.5	14.1	13.9
4	25	19.0	19.4	18.0
5	32	22.9	22.0	17.7
6	40	19.0	17.6	16.5
7	33	21.8	18.6	17.5
8	33	25.2	21.4	20.2
9	40	20.1	16.6	15.1
10	20	25.7	19.2	16.1
11	13	25.0	21.0	17.3
12	22	19.8	17.2	18.4
13	19	24.7	18.2	17.0
14	22	23.9	20.0	17.8
15	24	23.2	18.4	18.1
16	50	32.1	18.9	15.8
17	40	29.4	19.7	15.5
18	40	23.9	20.4	17.8

Panel side 2

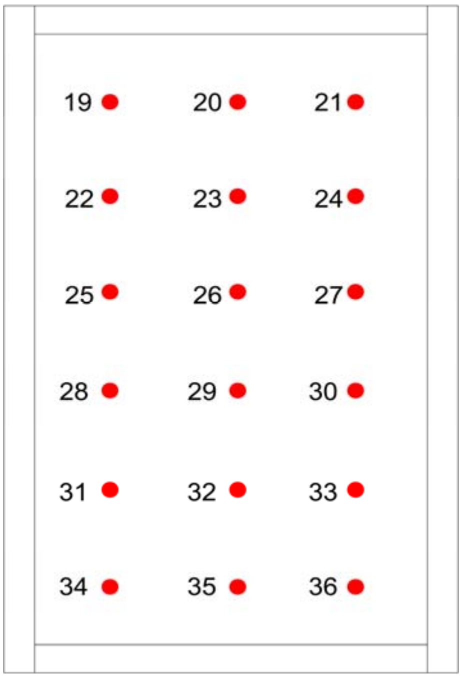

Figure 5.37 Render investigation locations side two

Table 5.7 Vertical compression panel render thickness– side two

Location	**Render Thickness (mm)**	**Location**	**Render Thickness (mm)**
19	17	28	40
20	22	29	36
21	28	30	23
22	35	31	37
23	30	32	29
24	38	33	37
25	10	34	20
26	17	35	18
27	25	36	10

Assuming a render thickness of 30mm the compressive stress at failure was 1.71 N/mm^2. The precise details about the render mix and water content are not known, but it can be safely assumed that they were similar to those used on the full size ModCell panels constructed for this investigation as the same type of render was used and the same renderer applied it. 1.71 N/mm^2 is 67% of the expected compressive strength of the render at roughly this age. The following reasons are proposed for the lower strength of the render. Firstly the render may have has a weakness or been slightly thinner at the location where it failed. Secondly the panel had previously been used to test the effects of continual exposure to the weather. When the panel was dismantled after testing it was found that where the render was thinnest there had been some decay of the straw and as a result the render was not adhered to the straw as well. This could have allowed for some buckling of the render as it was not being restrained by the straw. Finally the panel had been transported several times which may have caused small cracks in the render.

5.4.2 Conclusions

The load carried by the straw and render (51.2 kN/m) is comparable to the results achieved in work by Faine and Zhang (2002) when using similar constructions. They did not confine the render on their test specimens and therefore recorded much higher deflections. However as already mentioned, previous work was primarily concerned with the load carrying capacity for load-bearing straw bales. The focus of this test has been to see the effects of displacement of the straw and render. It has been shown in this test that the straw and render can be compressed by more than 10 mm over a height of 1.9 m (0.5% of wall height) before the render surface fails. It has also been shown that at the same time the straw and render can provide more than 50 kN/m resistance to vertical loading.

From this test we can conclude that on a full size ModCell panel the deflection at the centre of the top frame element must be less than 10 mm in order to avoid any damage. For a typical 3.2 m long ModCell panel the design deflection limit of the top of the timber frame would be L/360 if it had no brittle finishes. L/360 for a 3.2 m beam is 8.9 mm. If the top of the frame was designed as a beam, even under the most generous deflection limits its deflection would be less than 10 mm. Therefore there is no risk of damaging the render if other elements of the panels are designed adequately.

6 Computer Modelling of Racking Shear

6.1 Introduction

A computer model to predict the racking shear performance of the ModCell panels was developed during this investigation. It is hoped that the model will allow for design changes to be made and tested very easily and at minimal cost. If these changes produce promising results in the model then full scale testing may be considered. It is also hoped that the model will allow greater understanding of how ModCell panels behave structurally when being loaded. The modelling and analysis was carried out using Robot Millennium v 17.5.0.1764 (2004).

6.2 The design of the computer Model

The computer model was initially designed in two dimensions to allow ease of modelling and quick development. As only in plane loads are being considered a two dimensional model is perfectly adequate. Additionally three dimensional structures can easily be created from two dimensional panels, so the capabilities of the model are not restricted by starting off in two dimensions.

The two dimensional model of a single panel consists of the frame, render, bracing and reinforcement. The frame was modelled using simple bar elements. The corner joints in the frame have rotational stiffness and vertical stiffness derived from the laboratory testing which differ in opening and closing. Between the frame and the render there are elements with very low stiffness that represent the shrinkage gap between the render and timber frame on the real panels. The vertical reinforcement bars were modelled using simple bar elements. These were pin jointed at each end to the timber frame. The corer bracing was modelled in the same way. The render surface was modelled using a finite element membrane between the elements with low stiffness that represent the shrinkage gap. When the frame is loaded these elements will deflect and the render membrane will start to bear onto the timber frame. The finite element render membrane runs over the reinforcing bars and corner braces. No attempt was made to separate it from these elements.

Figure 6.1 shows an image of the model. Note that only the corner bracing that is put into tension when the panel is loaded has been included. In the real ModCell panels the bracing is fixed in such a way that it cannot be loaded in compression.

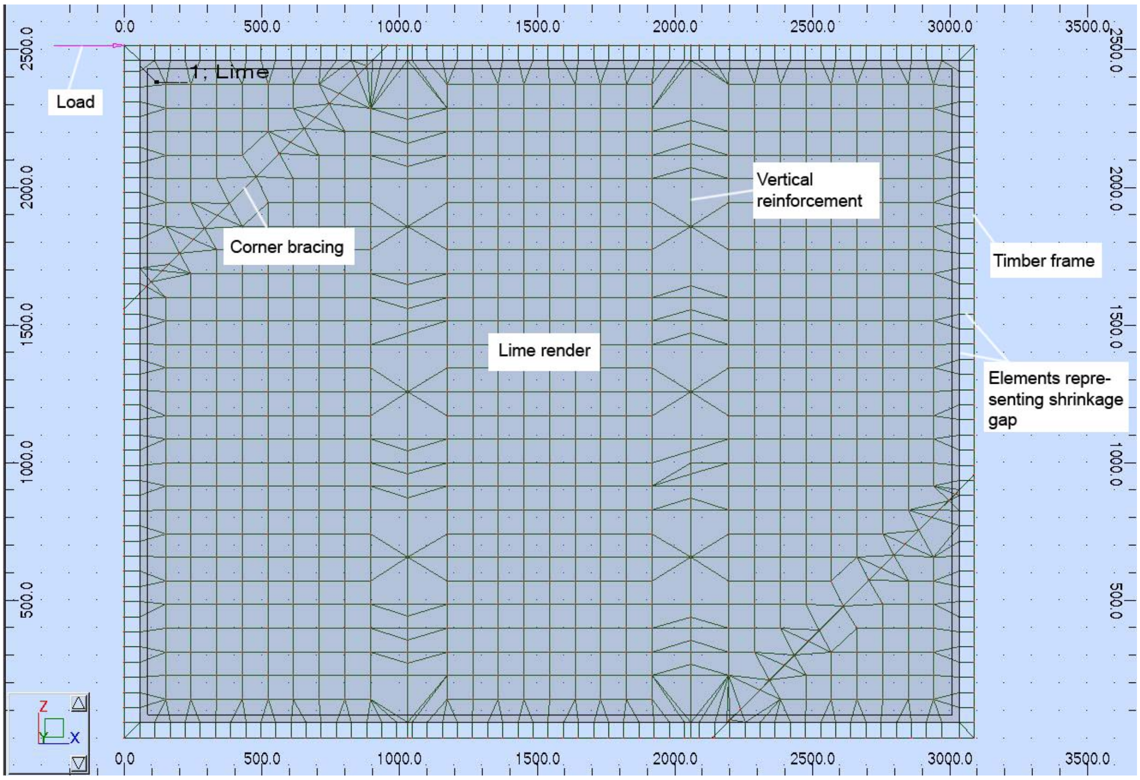

Figure 6.1 Basic two dimensional model in Robot

Details of how the model was constructed within the Robot millennium modelling software are shown in Table 6.1. As the model is two dimensional the cross sectional areas of the reinforcement and render have been combined from both faces and applied on one plane. All the nodes and elements in the model are set along the centroids of the elements in the real panel. The panel is supported continuously along its base. Only the corner braced panels were modelled as this type of bracing was considered most suitable for load-bearing panels following the structural testing.

Table 6.1 Computer model construction

	Element type	Cross section	Material properties
Timber frame	Simple bar	490mm deep x 100mm high	$E_{parallel}$ = 11000 N/mm^2 Shear Modulus, G=690 N/mm^2 Bending strength = 24.0 N/mm^2 Axial tension = 14.0 N/mm^2 Transverse tension = 0.5 N/mm^2 Axial compression = 21.0 N/mm^2 Transverse compression = 2.5 N/mm^2 Shear = 2.5 N/mm^2
Corner bracing	Simple bar	Area equivalent to two 12 mm diameter bars	E = 200000 N/mm^2 Poisson ratio, ν = 0.3 Shear modulus, G=76923 N/mm^2 Calculation strength=200 N/mm^2
Vertical reinforcement	Simple bar	Area equivalent to two 10 mm diameter bars	As corner bracing
Lime render	Panel - FE mesh using Coons method with 4 node quadrilaterals	60 mm thick	E = 5000 N/mm^2 Shear modulus, G = 2000 N/mm^2 Calculation compressive strength = 2.0 N/mm^2 (from laboratory testing)
Shrinkage gap	Simple bar		E = 0.01 N/mm^2 Poisson ratio, ν = 0 Shear modulus, G = 0.01 N/mm^2 Calculation strength = 500 N/mm^2

Both the rotational stiffness and vertical stiffness of the joints in the frame were derived from the laboratory testing discussed in Section 4 of this thesis. The screw connected joints were used. Figure 6.2 and Figure 6.3 show the stiffness information input into Robot in order to model the joints.

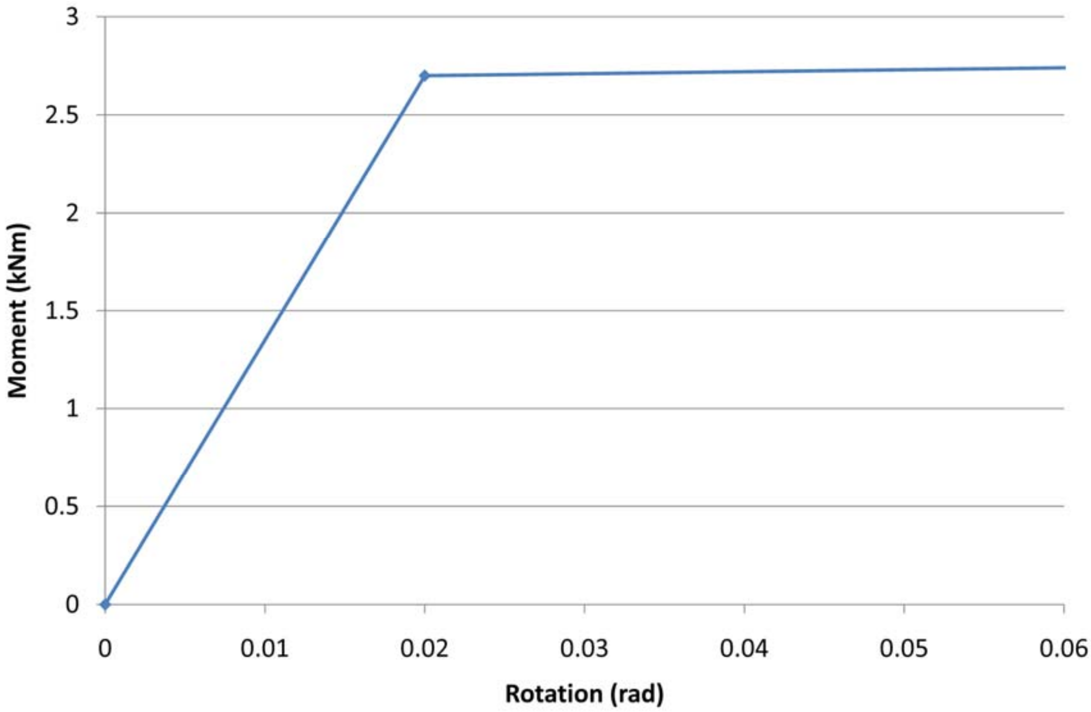

Figure 6.2 Robot model rotation joint stiffness data

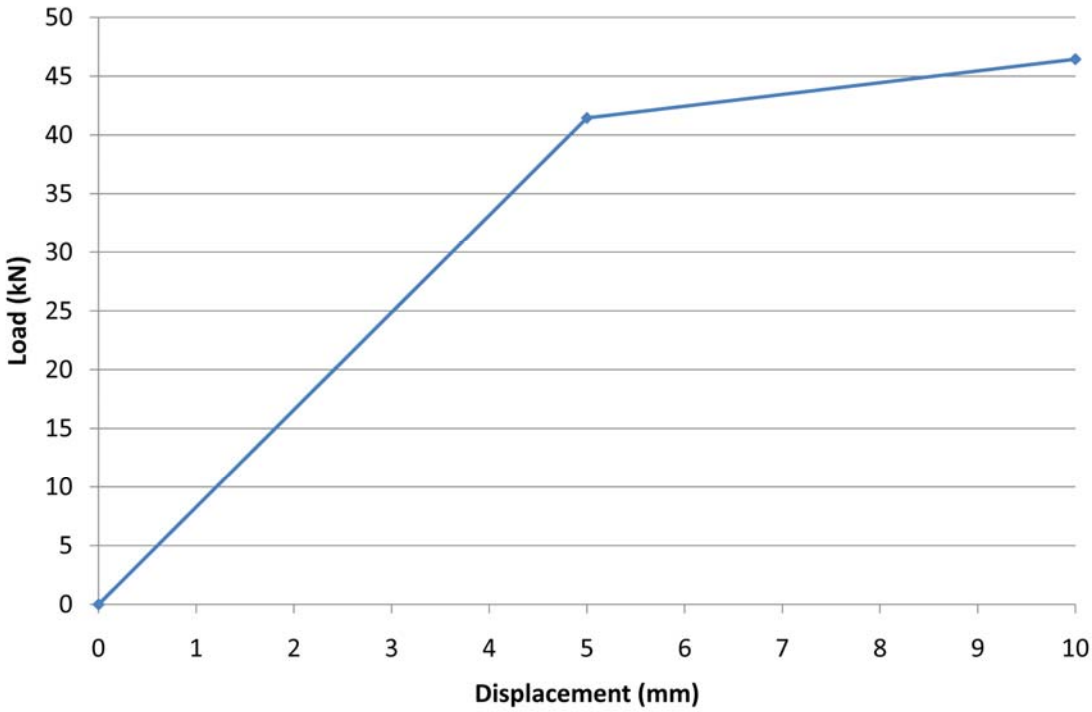

Figure 6.3 Robot model vertical joint stiffness data

The model is linear elastic and does not allow for material failure. Therefore it can only be used to predict the structural behaviour of the panel prior to material failure. This is not a limitation as the controlling factor in the design of load-bearing ModCell panels is the deflection under serviceability loading. From the panel racking shear tests in

Section 5.3 of this report it has been shown that at the serviceability deflection limit of h/500 there is no material failure. It may be necessary to develop a model which allows for the failure of materials, but this is a complex issue and is beyond the scope of this investigation.

Initially the model did not have the elements to represent the gap between the render and the timber, but this caused problems. Because the model does not have any failure criteria in it, only the material stiffness', when the panel was loaded the timber frame instantly loaded the very stiff render. This caused the whole panel to be over 100 times stiffer than the real panels. Therefore the gap was introduced into the model.

After the model has been run with set loads applied the following results can easily be obtained from it:

- Deflections
- Stresses in elements (timber frame, reinforcement, bracing)
- Stresses in render surface

Therefore it is possible to manually check the stresses in elements in order to establish if they are nearing failure.

6.3 Validation of the computer model

Once the basic models of the two bale and three bale corner braced panels were constructed they were validated and refined against the laboratory test results for the same panels. It was not possible to validate the generic design of the model against other laboratory tests carried out by Lawrence et al. (2009a) as the global rotation of the panels was not measured in these tests and therefore the results could not be corrected for this rotation.

For the validation of the models the load was applied in the model to the top corner of the panel as in the laboratory based racking shear tests. However in the model this loads both the top and side frame elements at the same time unlike in the laboratory based testing where only the top element is loaded and the load is transferred through the joint. In the model the load was applied in increments of 10 kN from 0 kN up to 60 kN. This load range is similar to the range that was applied during laboratory testing.

Figure 6.4 shows the results from the computer models of the two and three bale corner braced panels. Also included in Figure 6.4 are the results for the same panels from the laboratory testing. The graph plots load in kN/m against horizontal displacement at the top corner of the panel opposite to where the load is being applied.

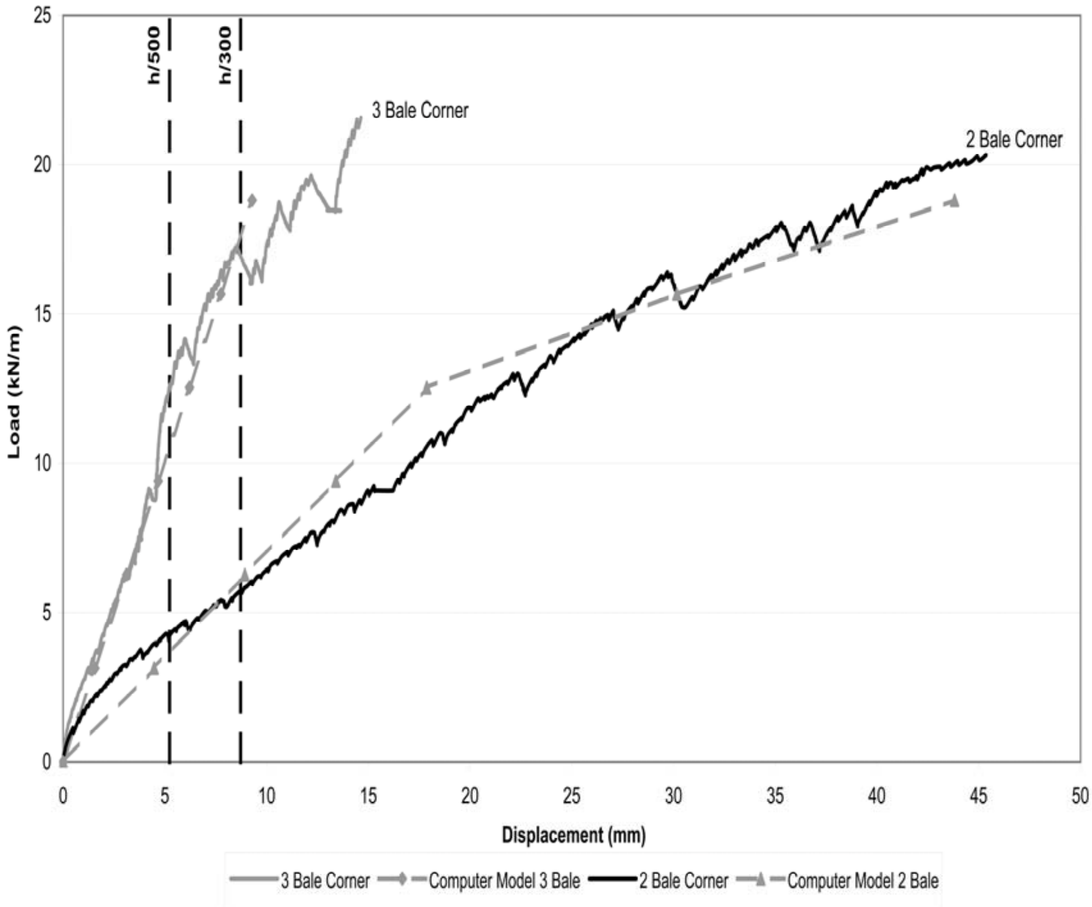

Figure 6.4 Validation of Computer models against laboratory test results

From Figure 6.4 it can be seen that the models predictions match the actual laboratory results well. The three bale panel model predicts the performance of the panels well up to the serviceability deflection limit of h/500. Beyond this point the stiffness of the actual panel begins to decrease before the first crack in the render at a displacement of h/280, while the stiffness of the model remains constant. This is due to the reasons mentioned in Section 6.2 of this report relating to the linear elastic nature of the model and the fact that the materials do not include non-linearity or failure criteria.

As with the three bale panel the two bale panel model predicts the performance of the two bale panel well up to displacements of around h/300. Following this the stiffness of the panel starts to decrease, as the render cracked at h/220, and the results from the laboratory testing and the model drift apart. However unlike the three bale model the two bale model predicts a change in stiffness at a load of 12.5 kN/m. There is also another very slight change in stiffness at a load of 15.7 kN/m. These changes in stiffness are due to the bilinear rotational and vertical stiffness' of the joints in the model. Because of this the model predicts the performance of the two bale panel well over the entire load range, slightly over predicting its strength at around 8 to 12 kN/m and under predicting at the end of the load range.

The three bale model did not exhibit these changes in stiffness as the panel is much stiffer overall and therefore the joints do not reach the required rotation of 0.02 radians for the stiffness to change (see Figure 6.2). However in the actual panels it is not only the change joint stiffness that causes the changes in stiffness of the whole panel. This is largely due to the render crushing and cracking. Therefore even though the two bale model appears to predict the performance well across the entire load range it is in fact doing this solely based on the changing joint stiffness.

Within the serviceability displacement range both models are sufficiently accurate and therefore very useful to this investigation and the future development of load-bearing ModCell panels. It would have been preferable to validate the models against further laboratory tests, but that was not possible within the remit of this investigation. Further laboratory tests will be under taken in the continuing development of ModCell panels, so it may be possible to utilise the results from those to further validate the models following the conclusion of this investigation.

6.4 Parametric Analysis

A parametric analysis was carried out on the three bale panel model in order to see the effects on the structural performance when certain elements were changed. During this the following were changed and analysed one at a time:

- frame thickness
- render thickness
- layout of vertical reinforcement
- layout of bracing
- fixity of the joints

6.4.1 Frame Thickness

Currently ModCell panels have 100 mm thick laminated timber frames. The model was run with frame thicknesses of 80, 100, 120 and 140 mm. The model was also run with the top element of the frame 160 mm thick with the sides and base kept at 100 mm. This is because White Design and Integral Structural Design are currently designing a building with ModCell panels with these frame dimensions. The horizontal displacement results from these analyses are shown in Figure 6.5.

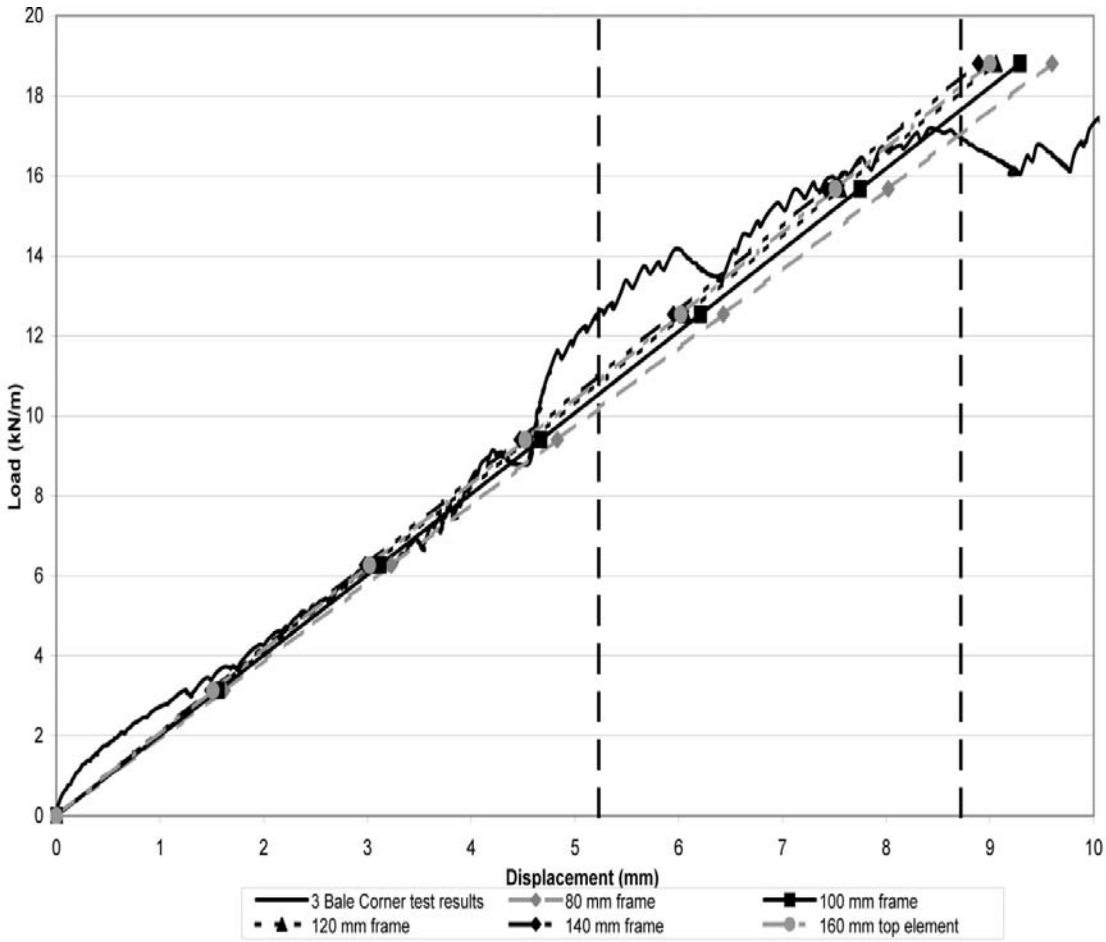

Figure 6.5 Results when frame thickness changed

From Figure 6.5 it can be seen that changing the thickness of the timber frame does make some difference to the stiffness of the panel. Table 6.2 shows the load carried by each panel at a displacement of h/500.

Table 6.2 Load at h/500 and stiffness with different frame thicknesses

Frame Thickness (mm)	Load at h/500 (kN/m)	Stiffness (kN/mm)
80	10.19	6.17
100	10.54	6.41
120	10.80	6.58
140	11.01	6.71
160 (top element only)	10.89	6.62

These results are as one would expect with the stiffness increasing as the thickness of the timber frame increases. However the stiffness of the panels is not solely derived from the timber frame. With a 40% increase in frame cross sectional area only a 5% increase in panel stiffness is achieved. By thickening the top of the frame to 160 mm the stiffness increases by 3%. This will be largely due to the way that the corner braced panel transfers its load through the bracing and frame, utilising the stiffness of the timber. This was previously discussed in Section 5.2.2 of this report.

6.4.2 Render Thickness

The thickness of render currently used on ModCell panels is 30 mm. For the next stage in the parametric analysis the render thickness was changed to 0, 10, 20, 30 and 40 mm and the model was re-run for each case. All of the other elements in the model were kept the standard ModCell construction. Figure 6.6 shows the horizontal displacement from these tests and Table 6.3 shows the load at h/500 and the stiffness.

Table 6.3 Load at h/500 and stiffness with different render thicknesses

Render Thickness (mm)	Load at h/500 (kN/m)	Stiffness (kN/mm)
0	0.71	0.43
10	6.57	4.00
20	8.85	5.37
30	10.54	6.41
40	11.92	7.25

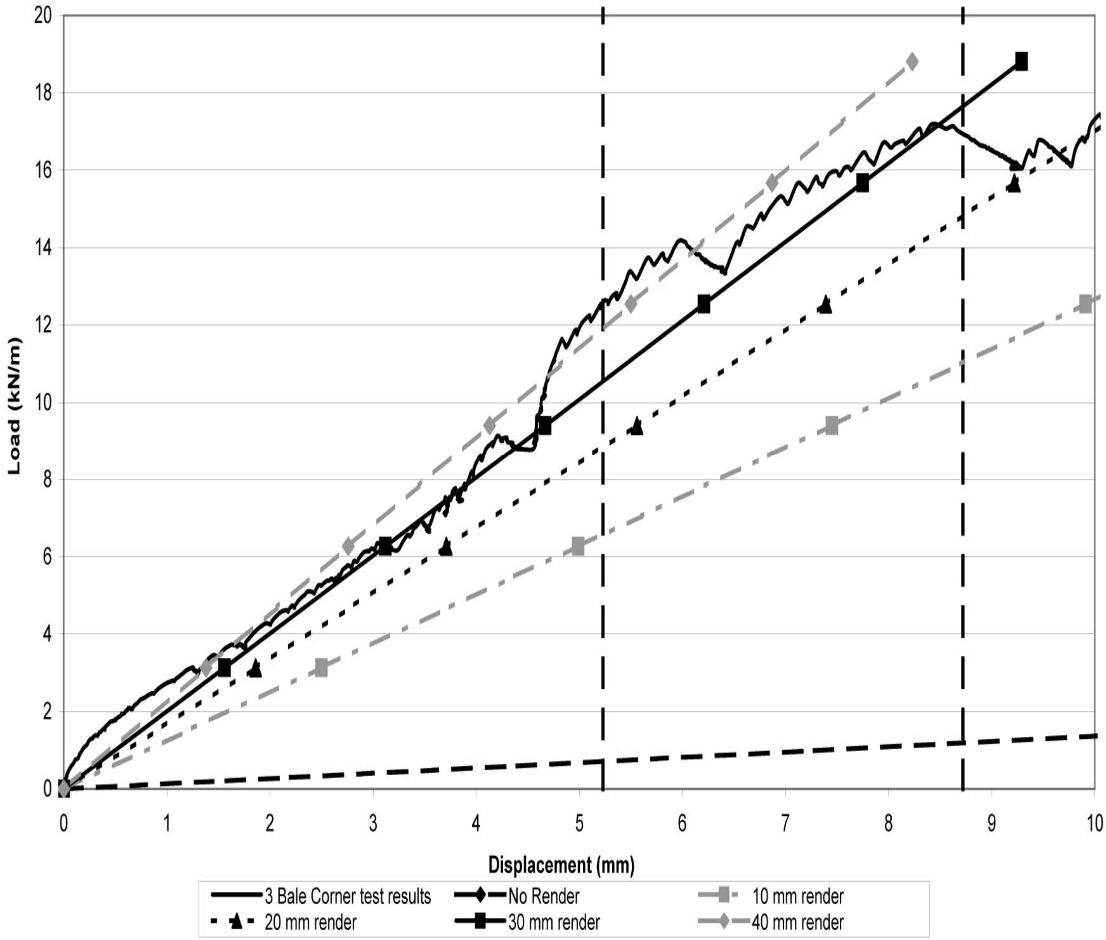

Figure 6.6 Results when render thickness changed

The changes in render thickness cause a larger change in the stiffness of the panel than when the frame thickness is changed. Lawrence et al. (2009a) observed that when a 30 mm render was applied to the panels they tested, their stiffness increased by 3.5 times. However the results from the modelling show that the stiffness increases 15 times when 30 mm of render are added. This is likely to be a result of the straw not being modelled, and therefore any stiffness gain from the straw within the un-rendered frame will not be present in the results. When there is no render the straw does add some stiffness to the panels as it fixed to the timber frame with timber stakes and therefore offers some resistance to shear. Once the render is added it is so much stiffer than the straw, that the effects of the straw become insignificant.

The other results in Figure 6.6 and Table 6.3 show that as the render thickness increases the stiffness of the panel increases. Comparing the results in Table 6.2 and Table 6.3 the addition of 10 mm of render to each face increases the stiffness of the panel more than when 40 mm is added to the thickness of the timber frame. The stiffnesses with 40 mm thick render and with 140 mm thick timber are 7.25 kN/mm and

6.71 kN/mm respectively. These results imply that thickening the render by 10 mm is solution for stiffening the panels. However both the timber frame and the render are costly elements in ModCell panels accounting for roughly a third of the total price each. Therefore, commercially, the additional stiffness offered by either may not be worth the increased cost. Another point to consider is the increased panel weight when 40% extra timber or 33% extra render is added. This would have implications on manufacture and transportation of the panels.

6.4.3 Reinforcement Layout

The computer model was run with the panel having zero, one, two and three vertical reinforcing bars in each face. Currently three bale ModCell panels have two vertical reinforcing bars in each face. In all of the models the bars were arranged evenly across the face of the panel. Figure 6.7 and Table 6.4 show the results.

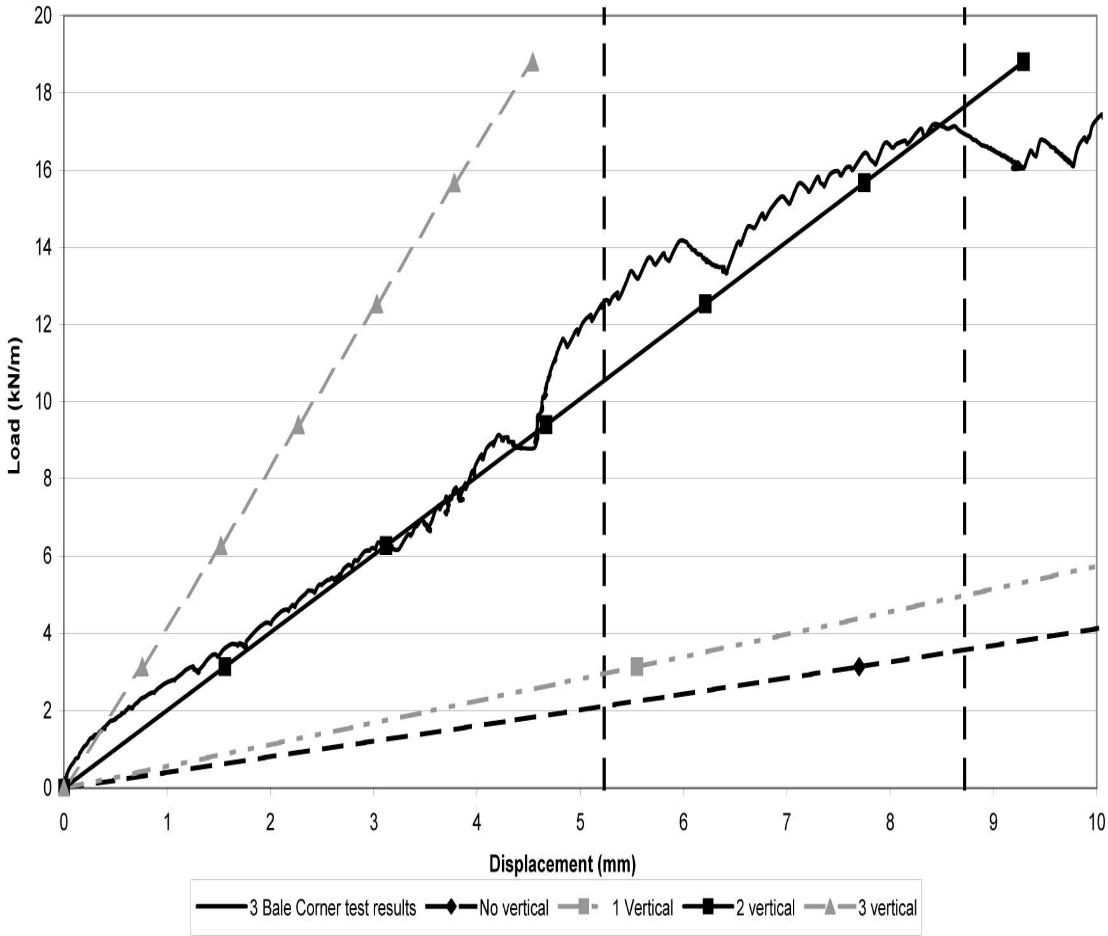

Figure 6.7 Results when reinforcement layout changed

Table 6.4 Load at h/500 and stiffness with different numbers of reinforcing bars

No. of reinforcing bars	Load at h/500 (kN/m)	Stiffness (kN/mm)
0	2.13	1.30
1	2.95	1.80
2	10.54	6.41
3	NA*	13.16
* At 60 kN load the deflection remained below h/500		

From Figure 6.7 it is clear that the number of reinforcing bars seem to have a large effect on the stiffness of the panel with the panel with three bars being twice as stiff as the panel with only two bars. The more vertical reinforcing bars there are within a panel the stiffer it should be as they provide some tensile reinforcement to the render and resist uplift of the top frame element. The results shown here suggest a large improvement in stiffness which may be an anomaly due to the way in which the computer model works. In the real ModCell panels a small strip of fibre mesh is placed over the reinforcing bars prior to rendering. This reduces the amount of adhesion between the render and the bars. The model does not allow for this and therefore the bars act as a composite together. Therefore these results should not be considered as definitive proof that adding a third vertical reinforcing bar will improve the stiffness of the panel by over 200%. Further computer modelling where the render and reinforcement are separated may provide further answers. Additionally full scale laboratory testing will confirm the effects of the reinforcing bars.

6.4.4 Bracing

The only change made to the bracing was to remove it in order to see what contribution it made to the stiffness of the panel. Lawrence et al. (2009a) tested individual joints with and without bracing and showed that bracing greatly increased the rotational stiffness of the joint in opening. Figure 6.8 and Table 6.5 show the results from modelling the panel with and without bracing.

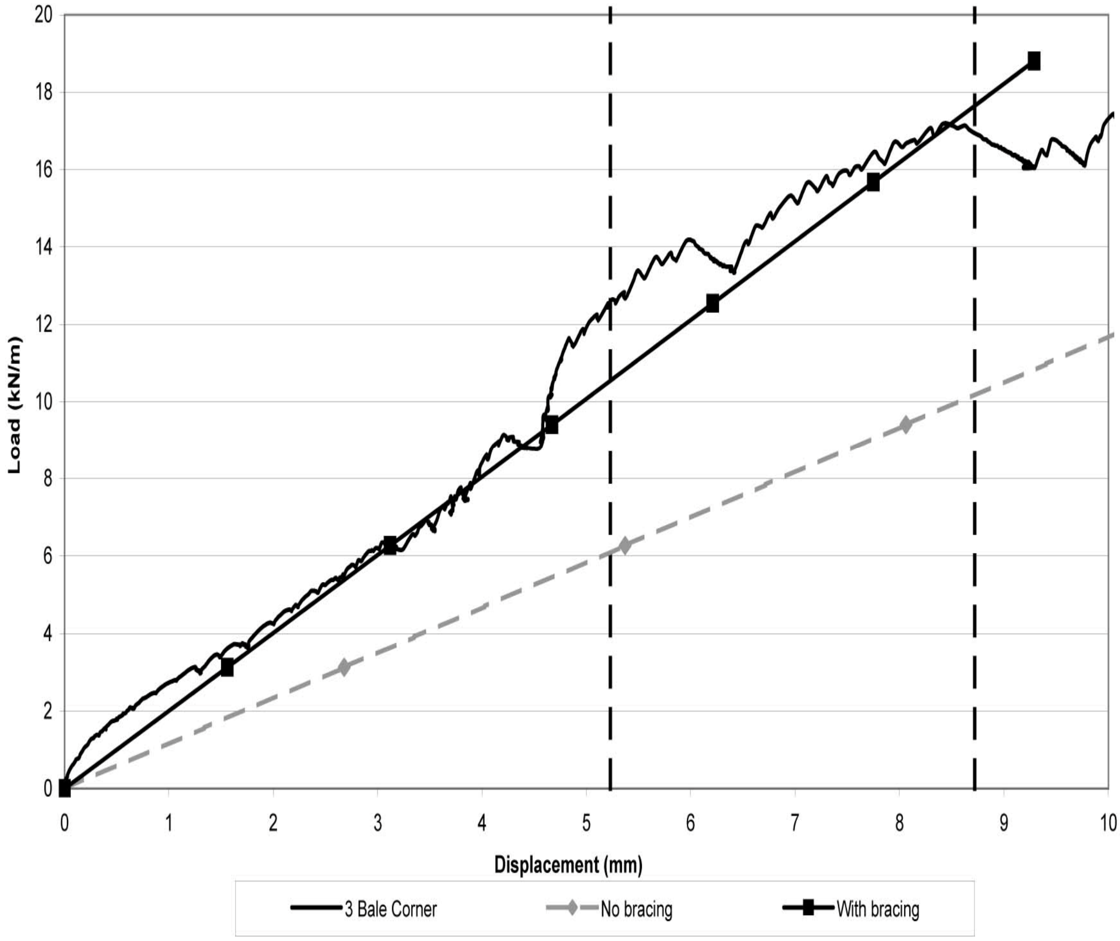

Figure 6.8 Results when bracing changed

Table 6.5 Load at h/500 and stiffness with and without bracing

Bracing	Load at h/500 (kN/m)	Stiffness (kN/mm)
With bracing	10.54	6.41
Without bracing	6.11	3.73

From these results it can be seen that the panel with bracing is stiffer, as one would expect. As previously mentioned Lawrence et al. (2009a) observed that bracing increased the stiffness of the joints they tested significantly. The effects of this can be seen here in these results with the braced panels having nearly twice the stiffness of the panel without bracing. Therefore corner bracing should be used in all load-bearing ModCell panels as it increases the stiffness at minimal cost and effort.

6.4.5 Joints

The three bale panel was modelled with fully fixed joints, pinned joints and 'normal' joints (joints with the stiffness's detailed in Section 6.2 of this report). While it is almost impossible to construct fully fixed joints and purely pinned joints, therefore meaning they would never be used on a real panel, it is interesting to see their affect on the stiffness of the panel. Figure 6.9 and Table 6.6 show the results from this modelling.

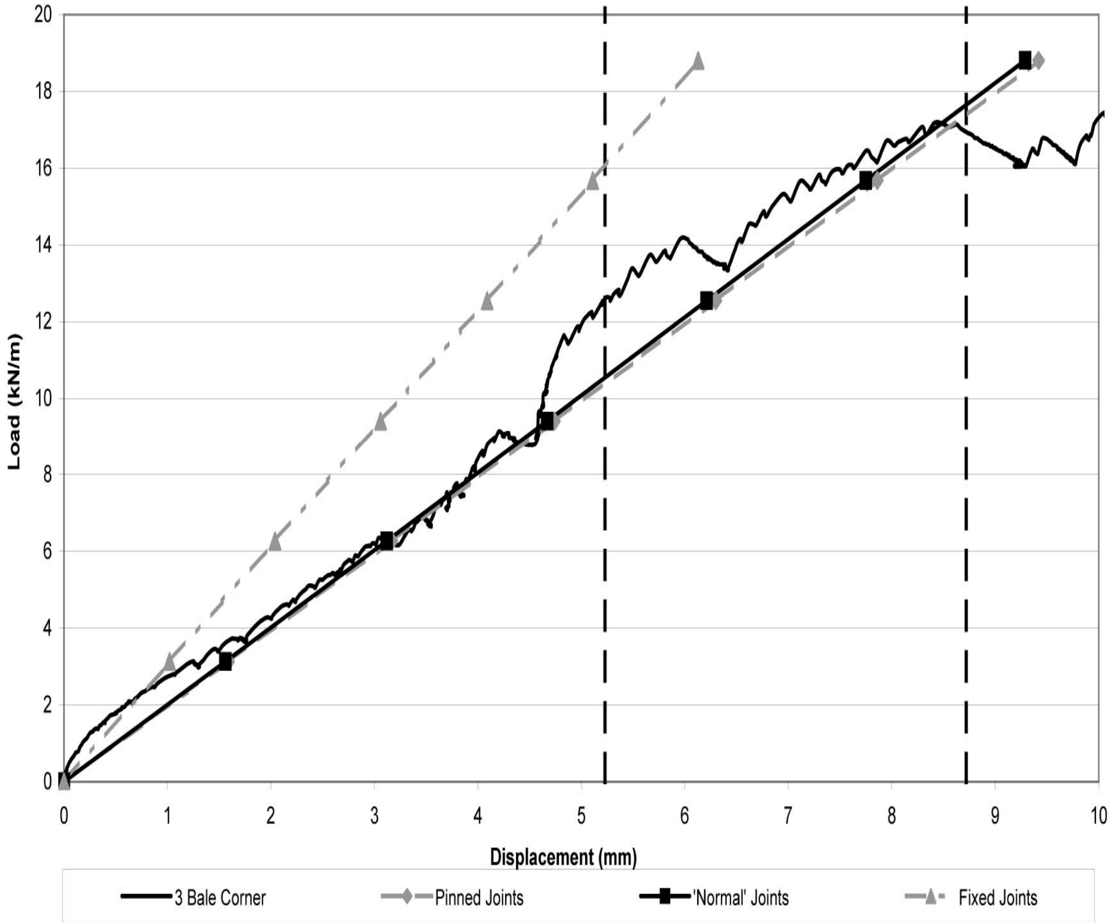

Figure 6.9 Results when joints changed

Table 6.6 Load at h/500 and stiffness with different joints

Joint	Load at h/500 (kN/m)	Stiffness (kN/mm)
Fixed	16.04	9.80
Pinned	10.39	6.29
'Normal'	10.54	6.41

Figure 6.9 and Table 6.6 show that fixed joints increase the stiffness of the panel by roughly 50%. This is a greater increase in stiffness than when the frame and the render

were increased in thickness. The pinned joint only reduced the stiffness of the frame by a very small amount when compared to the actual joints. This is because the pinned joint is pinned both rotationally and vertically. Therefore while it has no rotational stiffness it does have infinite vertical stiffness. The actual joint is semi-rigid and has some rotational stiffness and some vertical stiffness. As a result the pinned joint is rotating more under load than the normal joint, but it is not pulling apart vertically and therefore the results for both panels are very similar.

6.4.6 Further investigation of Joints

Richards (2009) investigated the rotational stiffness of ModCell panel joints as part of a final year undergraduate dissertation project. Three joint types were tested; a screw connected joint using washer head screws; a glued joint using standard head screws with glue between the joining surfaces; and a gusset plate joint using standard head screws and triangular plywood gusset plates across the joint. Details of the joints are shown in Figure 6.10. The joints were tested using the same method set out in Section 3.4.2 of this report. The results from the testing undertaken by Richards (2009) are shown in Figure 6.11 and Figure 6.12.

Richards (2009) comments that the gusset plate joints are considerably stiffer than the screw connected joints, but that failure is brittle and is likely to cause premature failure of the render when the gusset plate buckles. Also noted is the fact that the other joints have an ultimate strength limit between 5-6 kN in opening and 2-3 kN in closing, but that the glued joints are initially stiffer than the screw connected joints.

The stiffness data from the glued and gusset plate joint tests was used in the three bale panel computer model to assess the effects the different joint designs might have on the overall performance of the panel. As none of the joints were tested in vertical pull out the vertical stiffness of the joints in the model were kept as per the ‘normal’ joint (see Figure 6.3). The stiffness profiles for the glued and gusset plate joints used for modelling are shown in Figure 6.13.

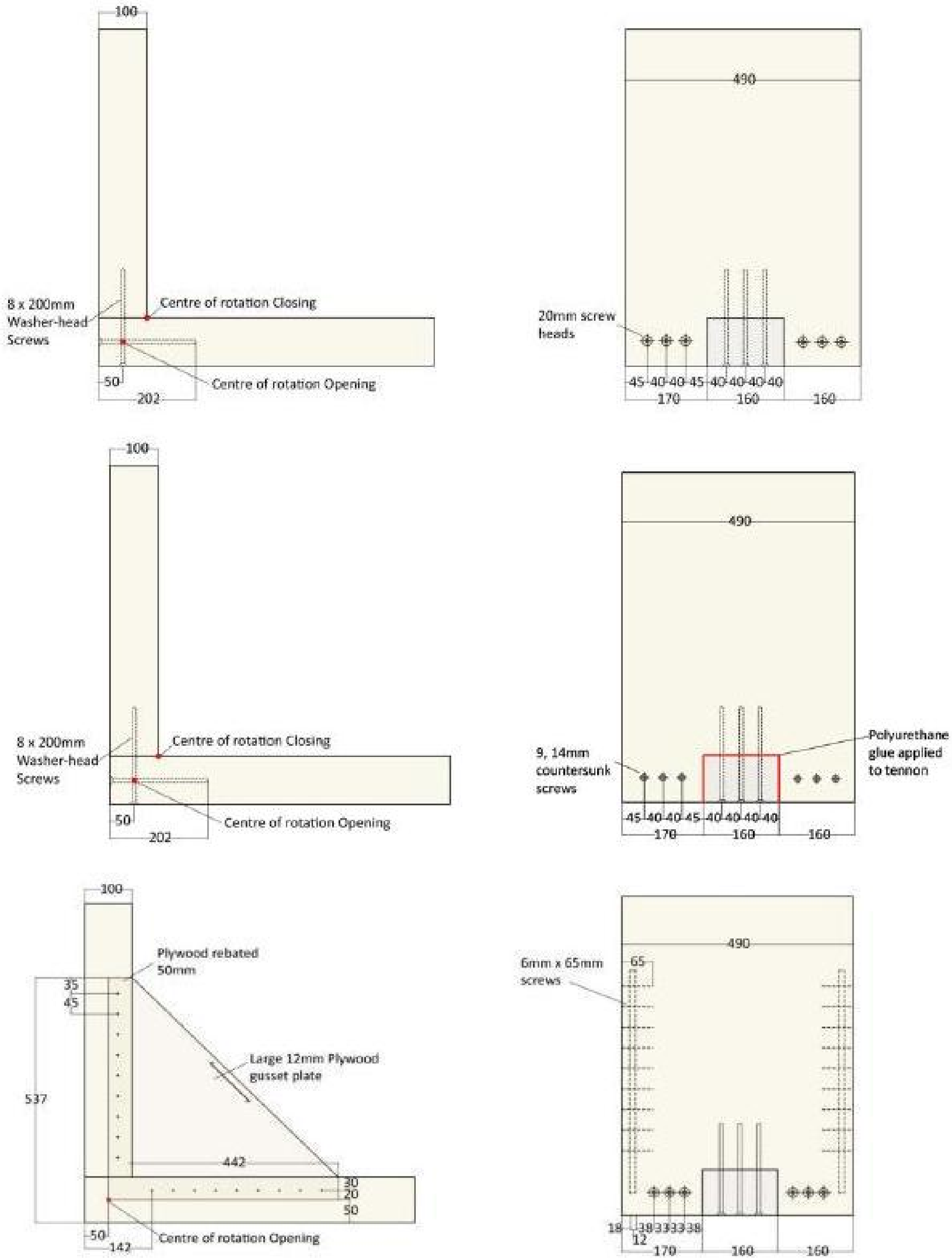

Figure 6.10 Joints tested by Richards (2009), Washer head screw (top), glued (middle), gusset plate (bottom)

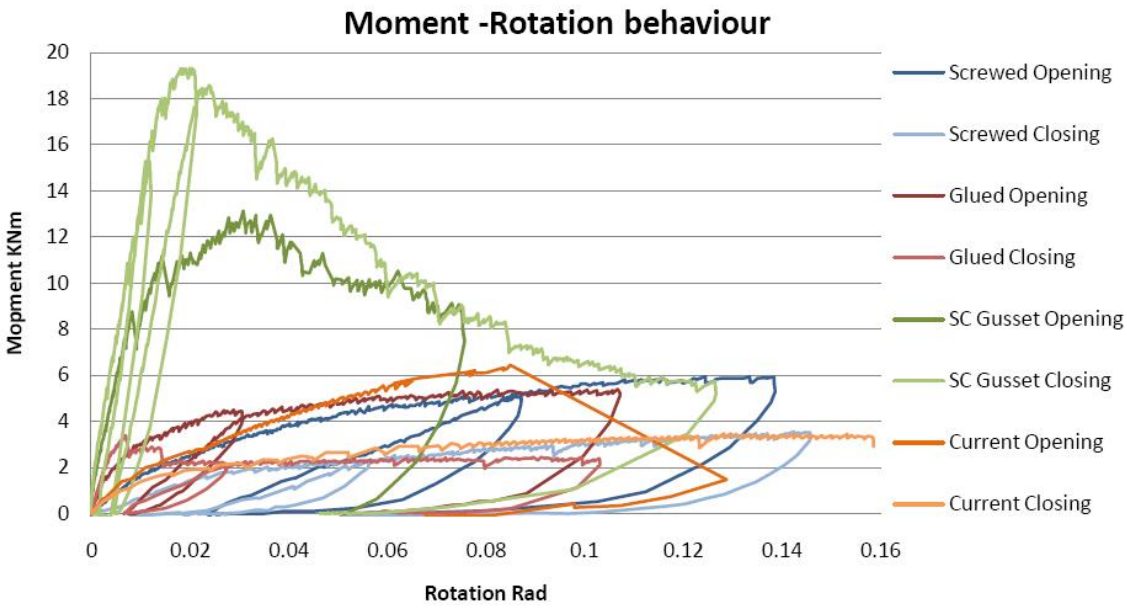

Figure 6.11 Results from testing by Richards (2009)

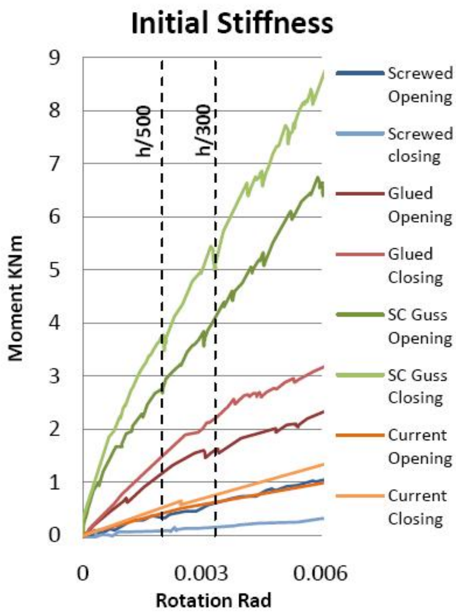

Figure 6.12 Results from testing by Richards (2009)

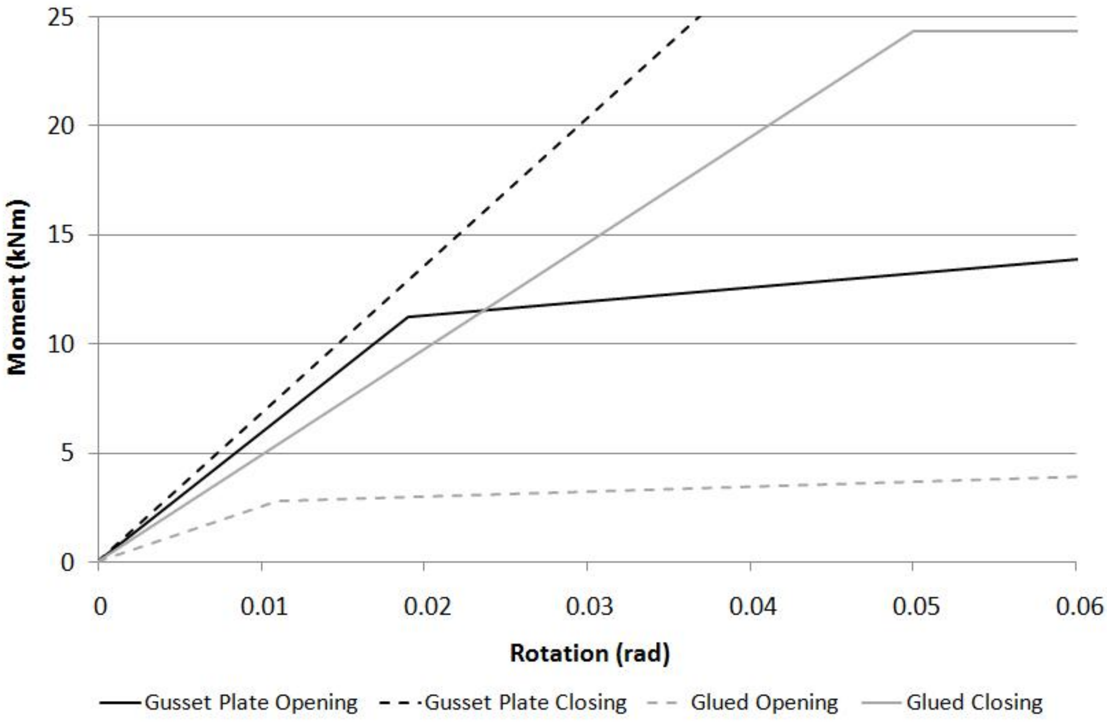

Figure 6.13 Glued joint opening stiffness profile

The model was run for each joint using the same loads as used previously. The model was also run using the gusset plate joint, but without any corner bracing in order to establish if it may be feasible to use the gusset plate as a replacement for the corner bracing. The results from this are shown in Figure 6.14 and Table 6.7.

Table 6.7 Load at h/500 and stiffness with Richards (2009) joints

Joint	Load at h/500 (kN/m)	Stiffness (kN/mm)
'Normal'	10.54	6.41
Glued	10.72	6.54
Gusset with bracing	11.61	7.04
Gusset without bracing	6.37	3.89
Fixed	16.04	9.80

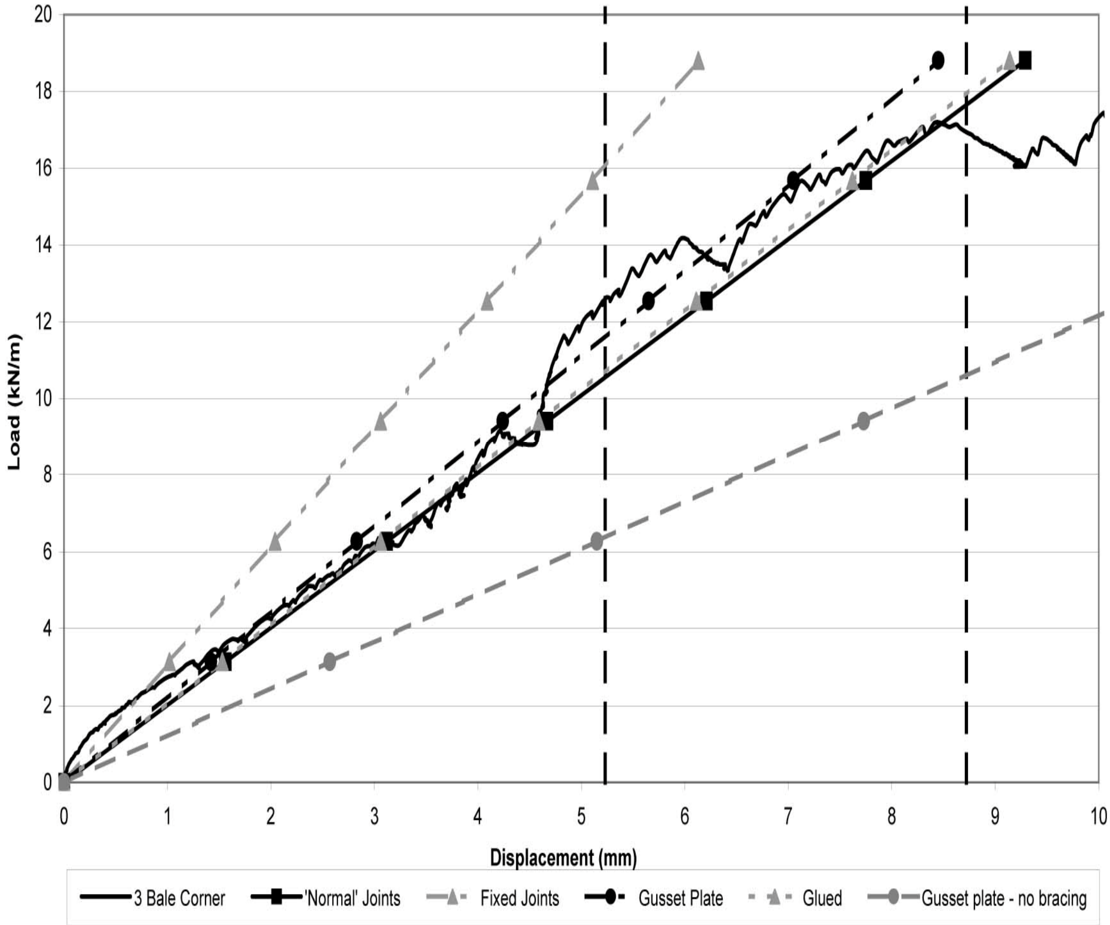

Figure 6.14 Results from simulations using Richards (2009) data

Figure 6.14 and Table 6.7 show some interesting results as generally the joints appear to have little effect on the stiffness of the panel. Only the fixed joint and the gusset plate without bracing caused any significant change to the stiffness. The rotational stiffness of the joints has little affect on the overall stiffness of the three bale corner braced panel as when loaded the joints do not rotate very much (previously discussed, see Figure 5.26). Therefore the stiffness of the corner bracing, timber frame and lime render has more effect. This is shown in the results from the panel with gusset plate joints, but no bracing. This panel only has 55% of the stiffness of the panel with gusset plate joints and bracing. This result also rules out the idea of utilising the extra stiffness of the gusset plate joint to allow the bracing to be removed as the panel will not be stiff enough to resist the required racking shear loads within the serviceability deflection limit.

Having previously mentioned that the joint stiffness has little impact on the panel stiffness, the theoretical fully fixed joints contradict this. This is because they are so much stiffer than any of the actual joints. However fully fixed joints are not possible to design or construct in reality when using timber with screw fixings and therefore should not be considered for use on load-bearing ModCell panels.

6.4.7 Summary of Parametric Analysis

From the parametric analysis it can be concluded that increasing the frame thickness or the render thickness increases the stiffness of the ModCell panel. The influence of the vertical reinforcement is slightly less clear as the increase in stiffness when using three reinforcing bars appears to have been exaggerated. However, it can reasonably be suggested that using three bars instead of two would increase the stiffness of the panel as the extra bar will further reinforce the render and increase the resistance to uplift of the top of the frame. The corner bracing adds greatly to the stiffness of the panel, but the corner joints do not.

From the results of this analysis a panel has been designed that incorporates the most suitable elements from the parametric analysis to create a stronger panel. Details of the panel design are below:

- 100 mm thick frame with 160 mm thick top
- 40 mm thick render
- Two vertical reinforcing bars
- Corner bracing
- Gusset plate joints

The majority of the frame has been kept at 100 mm as increasing this adds a lot of extra material to the panel. The top of the frame has been increased to 160 mm as this is a newly developed detail for ModCell panels and allows the first floor to be supported directly from the top of the panels. As shown in the analysis above it also adds some stiffness to the panel. The render thickness has been increase to 40 mm as this alone increases the stiffness by 13%. Two vertical reinforcing bars have been used as the results from this analysis were dubious; therefore they have been disregarded for this exercise. The corner bracing has been maintained as it adds significantly to the stiffness of the panels. Finally a gusset plate joint has been used as it increases the stiffness of the panels by nearly 10%. This panel was modelled Robot and the analysis run. The results are shown in Figure 6.15 along with the test results for the actual three bale corner braced panel and the model results for the same panel.

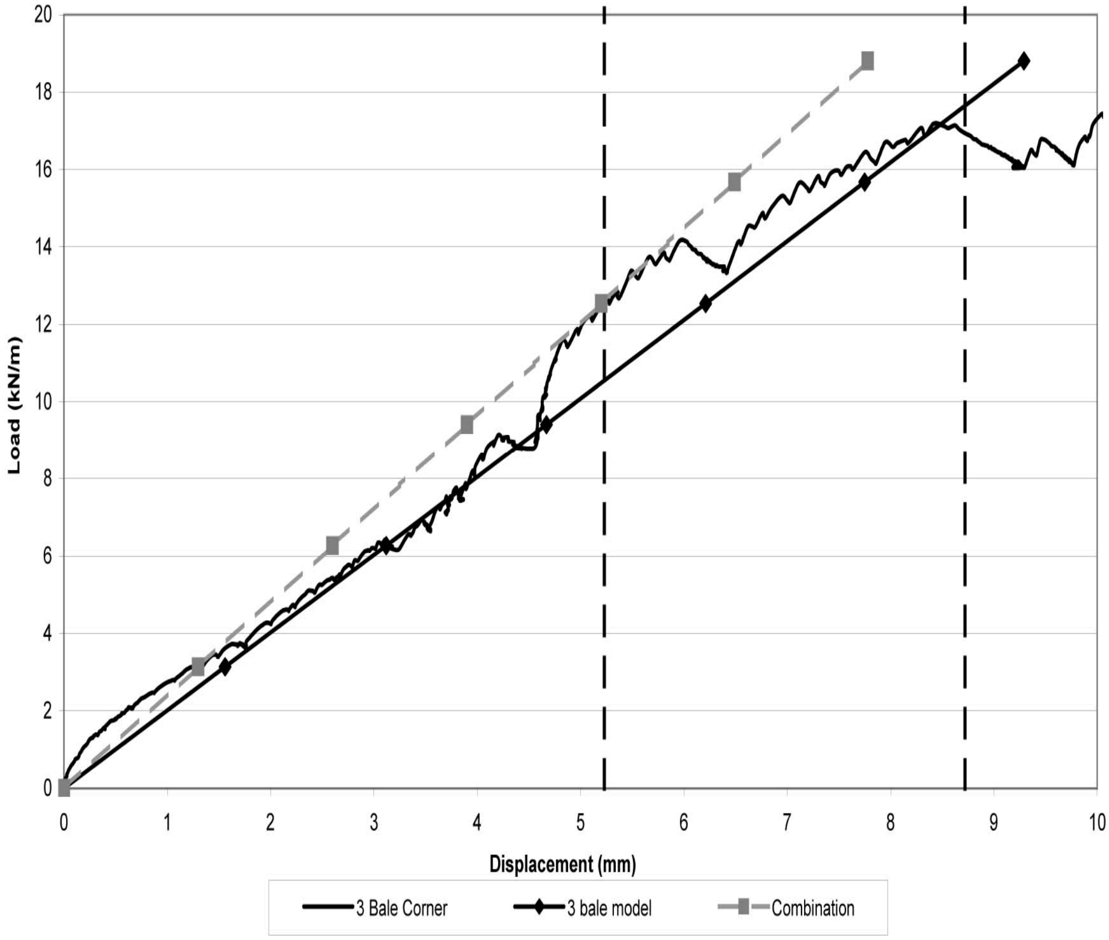

Figure 6.15 Results from model for improved panel design

The improved panel is 20% stiffer than the normal ModCell panel and it is able to resist over 2 kN/m additional load at the serviceability displacement limit of h/500. This is the type of application the Robot computer model has been developed for. It has shown favourable results for this panel design, and therefore it might now be considered for full scale laboratory testing. This however is beyond the scope of this investigation.

6.5 ModCell building at Bath

A full scale test building is being planned as part of a current ModCell research project at the University of Bath. At the time of writing the building is still being built. The building has been modelled using the Robot model previously discussed as it is a good opportunity to investigate how the model predicts the performance of entire wall panels and whole buildings in three dimensions. It is also a good opportunity to investigate how joining panels together affects their racking stiffness. The ground floor plan and an elevation for the proposed building are shown in Figure 6.16 and Figure 6.17.

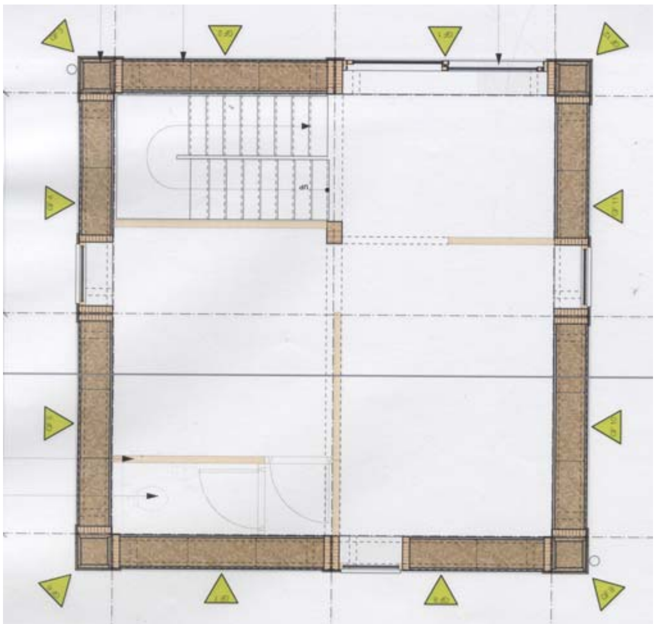

Figure 6.16 Ground floor plan of ModCell building at Bath

Figure 6.17 West elevation of ModCell building at Bath

Initially one elevation of the building was modelled and then from this the whole building has been modelled in three dimensions. The models are shown in Figure 6.18 and Figure 6.19. In a ModCell building the panels are joined together using plywood strips screwed across the joint between the panels. This provides an almost continuous connection along the panel edges. Therefore in the model the panels have been modelled fully fixed together. The three dimensional model is simplified as it does not include the first floor structure, but it is hoped it will be a good basis for future use of the model as a development tool.

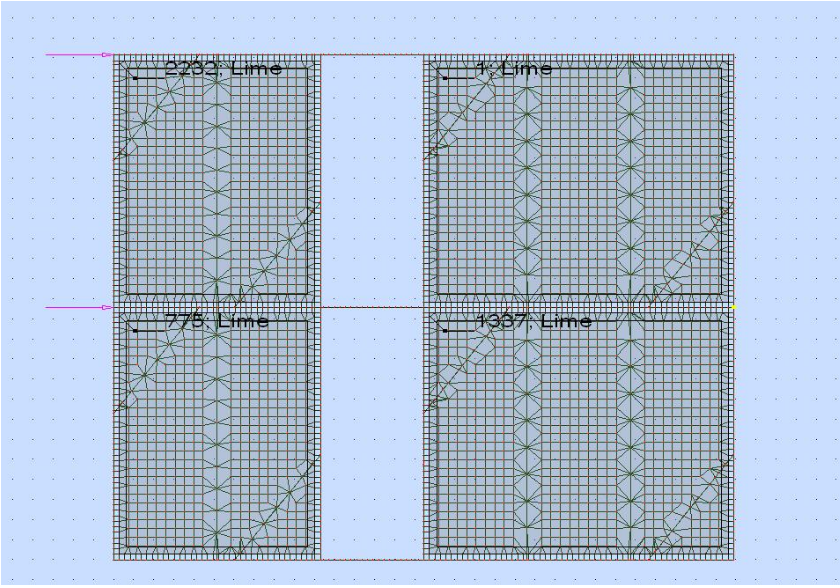

Figure 6.18 ModCell building model elevation

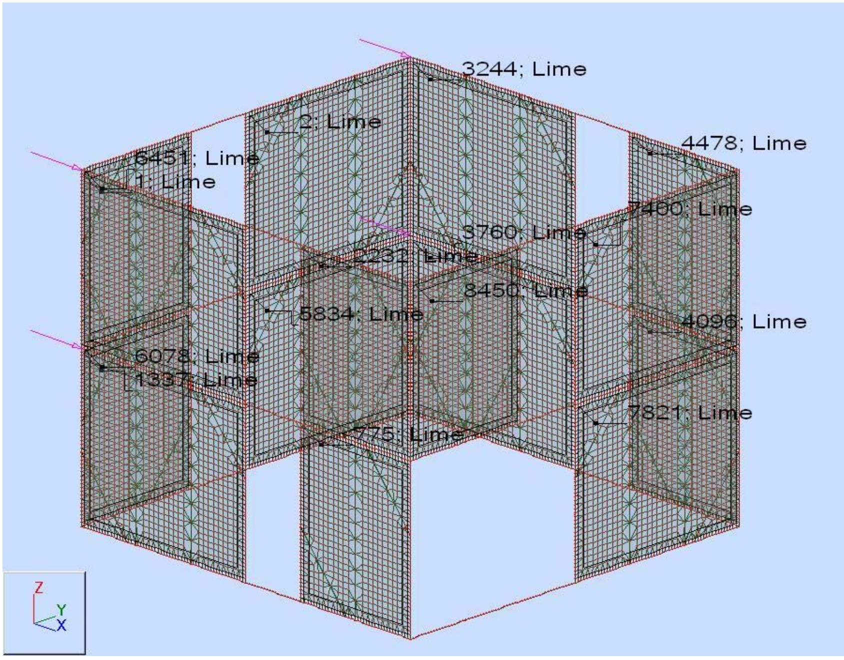

Figure 6.19 ModCell building three dimensional model

The elevation and the three dimensional model were loaded with the serviceability wind. For a building of this size the total wind load is in the order of 35kN. In the building this load will be carried by the two walls parallel to the direction of the wind. Therefore for the elevation the loading was halved to 17.5 kN applied as two loads of 8.75 kN, one at first floor level and one at roof level. For the three dimensional model the same load was applied to two walls parallel to one another bringing the total load up to 35 kN. The models were run and the results are shown in Table 6.8. Average deflections are shown for both first floor level and for roof level.

Table 6.8 Deflection and stiffness results from ModCell building model

Model	Average deflection at serviceability load (mm)	
	First Floor	Roof
Elevation	2.18	4.61
3D loaded in *x* direction	1.94	3.52
3D loaded in *y* direction	1.92	4.04

The serviceability deflection limits of h/500 for the first floor and the roof are 5.23 mm and 10.46 mm respectively. From Table 6.8 all of the deflections are well within these limits. The difference in the deflections in the *x* and *y* directions on the three dimensional model are due to different layouts of panels on each elevation. In the three dimensional model only the panels themselves were modelled. In the planned building, and also within other ModCell buildings, there are floors and internal walls which can be designed as shear walls. For example the shear walls can be constructed from solid timber or rammed earth and the effects on the buildings structural performance using these two materials will vary. However, all internal shear walls will stiffen the building and therefore the results displayed here are likely to be un-conservative.

The construction and then running of these models shows it is possible to develop the basic two dimensional model into more complex elevations and entire buildings. Ideally these results would be verified against testing of the actual structures. Unfortunately due to the scale this was not feasible during this investigation.

6.6 Future use of the model

The computer models have been developed in order to further understand load-bearing ModCell panels, but also to allow further development of ModCell panels. It is hoped that the model will be used by ModCell in their own development programme and also by future researchers working within the University of Bath who may potentially want to develop the model further as well as the panels. It is hoped that the model will allow for fast and more cost effective research and development as ideas can be trialled using the model and then if they produce promising results more costly laboratory testing can be undertaken.

6.7 Conclusions

Computer models of the two and three bale corner braced panels have been developed and successfully validated against laboratory test results for the same panels and therefore it can be said that the models are a success. However further validation against future laboratory testing would be useful in allowing further refinement of the models.

Parametric analysis has been carried out on the three bale panel model. This has shown that increasing the frame thickness or render thickness increases the stiffness of the panel. It has also shown that the corner joints have little effect on the stiffness of the panel, but that corner bracing does significantly enhance it. The parametric analysis has not shown the true effect of the vertical reinforcement on panel stiffness due to the way that the model was designed.

The models have proved useful in showing the potential increase in panel stiffness when the joints, render thickness and the thickness of the top of the frame are changed. The modelling of this panel design has shown the potential for a 20% increase in stiffness. Further investigation of this in the laboratory may now be considered, but it is beyond the scope of this investigation.

The modelling of both a wall elevation and entire three dimensional model of a future ModCell building has shown that the basic two dimensional model can be adapted and used to create more complex models.

7 Conclusions and Recommendations

7.1 Conclusions

Overall this research into the structural performance of load-bearing ModCell panels has been successful and provided some valuable results. The structural testing has proven the potential for ModCell panels to be used in load-bearing construction. Computer modelling has been carried out for the first time and has improved the understanding of how the panels behave structurally.

Before this investigation load-bearing ModCell panels had only been used once in a temporary building. Through this investigation it has been shown that it is possible for ModCell panels to be used in up to three storey load-bearing structures.

This investigation has built on the work by Lawrence et al. (2009a) where it was observed that the joints did not provide sufficient resistance alone to racking shear forces and that corner bracing greatly increased their stiffness. The parametric analysis of the computer model in this investigation has built on these findings by examining the effects of these and other elements of the panels to the overall stiffness of the panels.

Long term testing of the lime render strength has raised some interesting questions which currently remain unanswered. This is the first time long term testing has been carried out with this render and has shown the strength peaks at 28 days. Following this the strength drops off. The reasons for this are not yet fully understood and should be investigated further.

Joint testing had allowed the development of a suitable corner joint that does not fail during racking shear tests. The testing of other types of joint has proven that screw connected joints are the most suitable. Further testing on the screw connected joints has shown that both the screws in shear and in tension have an equal role in providing strength in the joint. This investigation has also shown that the screw connected joint strength can be calculated using Eurocode 5 by comparison of calculated and test derived strengths. Additionally the rotational stiffness of the screw connected joint has been found which has allowed the development of a computer model.

The panels from the Grand Designs Live house showed that transportation should be kept to a minimum in order to avoid excessive damage to the render. Testing of the panels showed how important build quality is to the structural strength of ModCell panels which is particularly important if they are to be used in load-bearing construction. The controlled demolition of the panels following testing showed the extent of water ingress into the straw and the damage this had caused. The damage occurred in a short space of time and therefore it is vital that ModCell panels are kept dry and covered before installation into a building.

Racking shear testing of four further ModCell panels has proven that the joints are now adequate and do not fail during racking shear. These tests also showed that using cross bracing does not offer any structural advantage over corner bracing. From the results of these racking shear tests alone the ModCell panels tested are stiff enough to be used in load-bearing construction at two and three storeys. These tests should however be repeated in order to verify the results.

The vertical compression test showed that the straw and render is strong enough to withstand loads of over 50 kN/m and vertical displacements of over 10 mm. This has proven that some deflection of the top element of the frames will be acceptable when loaded by a floor and therefore it is not necessary to use extra beams above the panels to support the floor. Additionally the results are comparable with those of Faine and Zhang (2002).

Developing a computer model of ModCell panels has already proven useful in determining the potential effect of changing the joint stiffness. It is hoped that the model will be developed further in the future and that it will allow a better understanding of how ModCell panels behave structurally.

A computer model of load-bearing ModCell panels has been developed and verified against the laboratory test results for the two and three bale corner braced panels. The model is accurate within the linear elastic ranges of the panels only as it does not allow for material failure. It is hoped that the model will aid future development of ModCell panels. This investigation has also shown that these basic two and three bale models can easily be compiled into entire elevations or buildings made from ModCell panels in order that analysis can take place in three dimensions.

Parametric analysis using the computer model of the three bale computer corner braced panel has shown the effects on structural performance of changing certain elements within the panel. It has shown that all elements within the panels have an effect on the stiffness of the panels, but that some have more effect than others. Interestingly it has shown, with the use of some work by Richards (2009), that the rotational stiffness of the joints has very little effect on overall panel stiffness. It has also been shown that by combining several changes the stiffness of the panel can be increased by 20%.

This investigation set out with the aims of improving the ModCell panels so that they could be used in two or three storey load-bearing construction and developing an accurate computer model that could predict their structural performance. Both of these aims have been achieved and have also raised some new questions that need to be answered. However, the future of load-bearing ModCell panels looks promising.

7.2 Recommendations for Further Work

All laboratory testing and investigation tends to answer many questions, but also raise many other questions and this investigation has been no exception. While full size racking shear testing has been completed on four panels, they were all of different designs. Therefore further testing on identical panels is recommended in order to fully verify the results. More test results will also help in further validating and refining the computer models.

The panels that have been tested have all been designed as load-bearing panels. They have not been tested for vertical load as it is known from design calculations following Eurocode 5 that they are sufficient. However the effect of vertical load on the racking performance has not been considered. This should be considered in further testing work as in a building there will always be some vertical load applied to the top of the panels. The effects of this upon racking shear resistance are unknown. It will reduce the uplift of the top element of the frame and therefore could reduce the displacements under racking loads, but additionally once the panel begins to rack there will be an eccentric vertical load which will have a tendency to increase the horizontal displacements.

Only single panels have been tested to date. Within a building the panels will be connected to their neighbours. This may have an effect on the stiffness of the panels and is therefore something that should be considered during further work. Other elements that are present in completed buildings and were not considered during testing are the first and second floors and internal walls. In current ModCell building design these are both solid timber panels and hence may offer considerable stiffness. This should be considered during further work.

Dynamic testing could also be considered during further work. Often the loadings structures are put under during their use are short term and dynamic. The racking shear loads applied to panels during this investigation are representative of wind loading which is very short term due to the sudden gusts of the wind. The effects of sudden loading and unloading should be investigated as if the deflections of the structure are too large then the occupants will feel the structure swaying which is undesirable and can be very disconcerting.

There are a couple of pieces of further work that could be undertaken on the computer models, although it is hoped that these will generally be used to further the development of the panels. Firstly they could be developed to include render failure. Even though the behaviour of the panels post render failure is not the most critical criteria in their design it would allow for investigation of this. Secondly the three dimensional model of the ModCell building could be further developed to include a first floor and internal walls. This would allow the effects of these to be further investigated.

Finally the effects of long term exposure of ModCell panels should be considered during future research. The Grand Designs panels tested during this thesis had been exposed to the weather when unprotected and had straw moisture contents above 25 % H_2O. Such high moisture contents are not likely to affect the short term strength of ModCell panels, however in the long term the straw will begin to decompose which will affect the bond with the render. The effects of long term exposure and its consequences should be investigated and measures put in place to minimise the potential for the panels to be subjected to the weather before they are installed in a building.

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